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A STUDY OF THE EFFECT OF TEMPERATURE UPON DIFFERENT REACTIONS AND PROCESSES

PRESIDENTIAL ADDRESS BY H. K. BARROWS,* BOSTON SOCIETY OF CIVIL ENGINEERS
MARCH 17, 1937

IT was found by Van't Hoff[1] (1852–1911) that the rate of activity of chemical reactions in general is multiplied by 2 or 3 for each rise in temperature of 10° C. Van't Hoff's law is often stated in general terms that many chemical reactions and physiological processes double in activity for every increase in temperature of 10° C. (or 18° F.). This is, however, only an approximation. The temperature coefficient of chemical reactions, *i.e.*, the ratio of increase per 10° C., is called Q_{10} and is commonly known as the Van't Hoff coefficient. Accordingly, if a chemical reaction was proceeding at rate 1 at 0° C., at 10° C. it would proceed at a rate two or three times as fast. At the usual temperatures for growth of plants — 20° to 30° C. — this same reaction would proceed at a rate from 4 or 9 to 8 or 27.

Van't Hoff[2] determined Q_{10} for some seventeen different chemical reactions, finding this varying between 1.2 and 3.68 and averaging about 2.6. He also determined Q_{10} for the respiration of certain plants (wheat, lupines and syringa), which he found to be 2.5.

---

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Arrhenius (1859–1927) also studied temperature effects and formulated a theoretical basis for comparison which may be expressed as follows:

$$\log K_t - \log K_o = \frac{\mu}{2.30\,R}\left(\frac{1}{T_o} - \frac{1}{T}\right)$$

where K_t and K_o are reaction velocity constants at absolute temperatures T and T_o, respectively (or the observed temperature in °C. plus 273.1°). With a log plot of ordinates and linear scale of abscissas $\left(\frac{1}{T_o} - \frac{1}{T}\right)$ this is the equation of a straight line with $\frac{\mu}{2.30\,R}$ as its slope.

For a given series of observations such a plot will give the slope C in the equation $C = \frac{\mu}{2.30\,R}$ (where R is the gas constant, 1.989) and hence $\mu = C \times 2.30\,R = 4.58\,C$.

The factor μ as thus determined is the temperature characteristic and will be expressed in calories per gram molecule of the reacting substance. Recently it has become customary to call this factor the energy of activation, using the symbol A in place of μ, and this notation will be used.

The relation between Q_{10} and A, which varies somewhat with temperature, may be determined as follows:

Since $Q_{10} = \frac{K_t}{K_o}$ and $\log Q_{10} = \log K_t - \log K_o$

and $\left(\frac{1}{T_o} - \frac{1}{T}\right) = \frac{T - T_o}{TT_o}$

$$A = 2.30\,R \log Q_{10} \times \frac{TT_o}{T - T_o}$$

$$= 4.58\left(\frac{TT_o}{T - T_o}\right) \log Q_{10}$$

For 0°–10° C. $A = 4.58 \times \frac{283.1 \times 273.1}{283.1 - 273.1} \log Q_{10} = 35{,}400 \log Q_{10}$

For 10°–20° C. $A = 38{,}100 \log Q_{10}$

For 20°–30° C. $A = 40{,}800 \log Q_{10}$

For 30°–40° C. $A = 43{,}500 \log Q_{10}$

In Fig. 1 are plotted the relations between Q_{10} and A for convenience in use.

Thus, for the Van't Hoff Q_{10} of 2, often used, A will, for temperatures between 0°–40° C., lie between 10,700 and 13,000, averaging about 11,800.

Considerable study of the Van't Hoff and Arrhenius temperature characteristics has been made, particularly, and more recently, in the fields of physiological chemistry and biology. It appears to be of value in hydrology and in the study of evaporation, transpiration and other water losses which are of importance as affecting the run-off of streams.

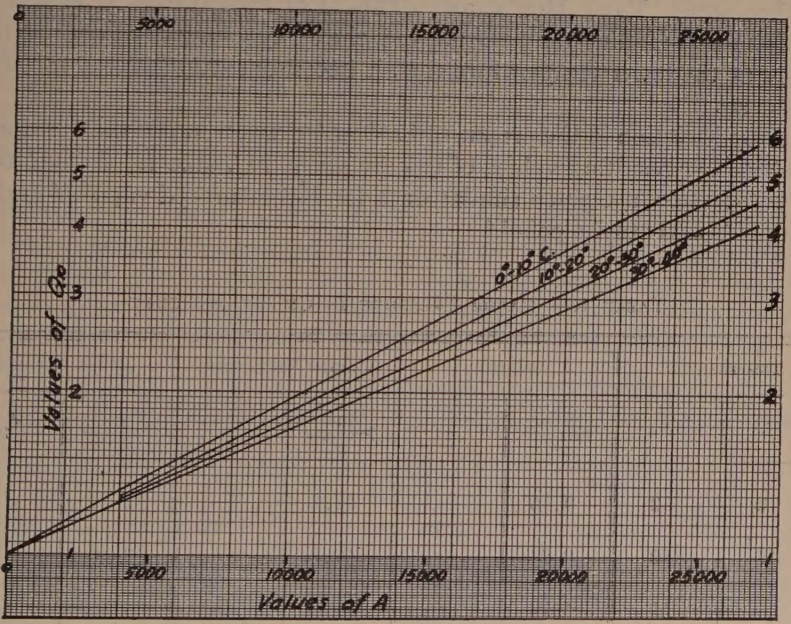

FIG. 1. — RELATION OF VAN'T HOFF Q_{10} AND ARRHENIUS A

The purpose of this study is to determine and examine the values of Q_{10} and A for a variety of processes and compare and relate these as far as this may be practicable.

PRESSURE OF SATURATED WATER VAPOR

The pressure of saturated water vapor is a factor of fundamental importance in hydrology as affecting precipitation and evaporation.

The atmosphere contains varying amounts of water vapor, depending upon its degree of saturation and temperature. At a given temperature, with complete saturation (or 100 per cent relative humidity) this water vapor has a fixed elastic pressure, which has been accurately determined experimentally. If the relative humidity is less than 100 per cent the actual vapor pressure in the air will be correspondingly reduced.

Variation in the pressure of saturated water vapor shows a very marked increase with temperature, following quite closely, as will be further noted, Van't Hoff's law. This is of importance in producing rainfall due to falling temperature and the air reaching the "dew point," where at the lower temperature the air becomes saturated and moisture condenses.

The variation in the pressure of saturated water vapor with temperature and the resulting values of Q_{10} are given in Table 1. In addition, for comparison, are given similar data for the saturated vapor of carbon bisulphide and ethyl alcohol.[3]

TABLE 1. — VAPOR PRESSURE IN CM. OF MERCURY

TEMPERATURE		WATER		CARBON BISULPHIDE		ETHYL ALCOHOL	
°C.	°F.	Vapor Pressure	Q_{10}	Vapor Pressure	Q_{10}	Vapor Pressure	Q_{10}
0	32	0.46		12.8		1.22	
			1.98		1.55		1.95
10	50	0.91		19.8		2.38	
			1.92		1.50		1.85
20	68	1.74		29.8		4.40	
			1.82		1.46		1.78
30	86	3.15		43.5		7.81	
			1.74		1.42		1.71
40	104	5.49		61.8		13.37	
Mean . .			1.87		1.48		1.82
A . .			10,600		6,700		10,200

As will be noted from Table 1, the variation of pressure of saturated water vapor with temperature follows the general statement of Van't Hoff's law ($Q_{10}=2$) quite closely, with Q_{10} varying from 1.98 to 1.74

and averaging 1.87, for which the Arrhenius $A = 9{,}500$ to 11,800 and averages about 10,600. The factor Q_{10} lessens somewhat as temperatures increase.

Carbon bisulphide vapor pressure approximates a value of $Q_{10} = 1.5$, and ethyl alcohol, $Q_{10} = 1.8$.

EVAPORATION FROM WATER AREA

Evaporation from water area, according to the law enunciated by Dalton, depends (in the absence of wind) chiefly upon the gradient between the pressure of saturated water vapor at water temperature and the actual vapor pressure of the air. It might therefore also be expected to be in harmony with Van't Hoff's law, as temperature is the important factor in respect to the variation of the vapor pressure gradient.

Data of observed evaporation from water area include, of course, wind effects, — an important factor, — so that only an approximation of values of Q_{10} in Van't Hoff's law can be derived from such observations.

Based upon observed data at some eight stations of yearly mean evaporation and temperature, the following has been determined:[4]

TABLE 2. — YEARLY MEAN EVAPORATION AND TEMPERATURE

TEMPERATURE		Yearly Evaporation (Inches)	Q_{10}	A
°C.	°F.			
0	32	17		
			1.95	10,200
10	50	33		
			1.54	7,200
20	68	51		
Mean		–	1.75	9,000

This phenomenon is therefore in fairly close agreement with Van't Hoff's law.

PHOTOSYNTHESIS OF PLANTS

Photosynthesis is the process of manufacture of carbohydrates in the leaf cells of green plants through the action of sunlight and in the presence of the chlorophyll. Because of chlorophyll a green leaf is typically a starch factory, but working only in sunlight. Plants without chlorophyll are either parasites or live on dead or decaying plants. Photosynthesis is basic to all plant life and hence animal life as well, and is the only process in nature by which living matter is made out of simple organic chemical elements with the aid of the radiant energy of the sun's rays. All vegetable forms except fungi are engaged in the conversion of CO_2 and water into sugars and starches. The leaves of the plant in their chief function carry on this process, the amount depending upon strength of light, amount of CO_2 or water present, etc. Equally as important as the absorption of CO_2 in photosynthesis is the production and liberation of oxygen.

The elements of water, H_2O, are assimilated in photosynthesis as well as carbon, and water is a requirement of the process.

The following statements by Spoehr[5] are of interest:

> "So far as the matter of food material is concerned there exists a closed cycle. Man feeds partly on animals and animals on plants; the plants feed on the carbon dioxide given to the air by animals as a result of the latter's use of food. Thus the plant reconverts the waste products of animal metabolism into food. The latter process is called photosynthesis."
>
> "All living things on the earth demand for their maintenance and propagation a continuous supply of energy. In final analysis plants are the fundamental source of energy of all animals as well as man.
>
> "It thus becomes evident that all the life on the planet depends upon the energy derived from the sun through the intermediary of the plant, *i.e.*, through the process of photosynthesis. Mankind lives entirely on the energy derived from the sun through the pursuits of agriculture."

Spoehr[7] gives values of Q_{10} for photosynthesis as determined by Warburg from experiments upon the unicellular alga Chlorella, as shown in Table 3.

TABLE 3.—PHOTOSYNTHESIS OF CHLORELLA

TEMPERATURE		Relative Light Intensity	Q_{10}	A
°C.	°F.			
5–10	41–50	16 45	4.7 4.3	23,700 22,400
10–20	50–68	45	2.1	12,300
16–25	61–77	16 1.8	2.0 1.06	11,800 1,000
20–30	68–86	45	1.6	8,200
25–32	77–90	1	1.0	0

In Table 3 is included the relative light intensity in each case, and, as will be noted, Q_{10} is approximately 1 with low light intensities and high temperatures.

This substantiates the view held by a number of investigators, that the process of photosynthesis involves two reactions, viz., (1) an ordinary chemical reaction, which determines the rate of the process at high illumination intensities and has a high temperature coefficient; and (2), a photochemical reaction which determines the rate of the process at low illumination intensities and has a low temperature coefficient. No satisfactory explanation appears for the high values of Q_{10} at low temperatures, although this appears to be a common occurrence in biological processes.

As will be noted from Table 3, between about 10°–30° C. (or 50°–86° F.) with high light intensities, Q_{10} approximates 2 (A = 11,800).

Van den Honert (quoted by Spoehr) also gives in a recent paper[8] for photosynthesis at temperatures between 12° and 20° C., with high light intensities, a $Q_{10} = 1.87$ ($A = 10{,}500$), but when light is a limiting factor $Q_{10} = \pm 1$ ($A = 0$).

In the process of photosynthesis where CO_2 is absorbed and as H_2CO_3 reacts with the chlorophyll the Van't Hoff Q_{10} is found to lie between 2 and 3 (A = about 11,000 to 18,000) according to Harvey,[6] although certain other stages he calls physical processes, and fixes Q_{10} at 1 to 1.4 ($A = 0$ to 5,700).

In leaves which are rich in chlorophyll the sunlight intensity can be reduced to one half or even one quarter the normal without slowing of the photosynthetic rate, showing that light intensity may exceed the maximum required. When light and CO_2 are not limiting factors, viz., in high light intensity, the so-called Blackman reaction occurs, and the photosynthetic process has a Q_{10} in the vicinity of 2, as shown in Table 4.[6]

TABLE 4. — PHOTOSYNTHESIS — BLACKMAN REACTION

TEMPERATURE		Rate of Blackman Reaction	Q_{10}	A
°C.	°F.			
10	50	100		
			2.30	13,800
20	68	230		
			1.74	9,800
30	86	400		
Mean		–	2.02	11,800

TRANSPIRATION OF PLANTS

The under surfaces of leaves of plants (and to some extent the upper surfaces) have numerous small openings called stomata, protected by so-called guard cells which serve to close the stomata. These guard cells become swollen and spherical after absorption of water brought up in excess into the leaves. In this condition, because of their structure, they fail to block effectively the small openings or stomata, and excess water in the leaves escapes.

The amount of water transpired by different plants has been segregated and determined experimentally by Briggs & Shantz and others.[9] Thus, for wheat a water requirement of 1,357 times the weight of wheat kernels produced was found, which would correspond to about 7 inches depth of water upon the cropped area during the growing season, with a crop yield of 20 bushels per acre.

Meyer[10] has concluded that transpiration follows Van't Hoff's law with $Q_{10}=2$ ($A=$about 11,800), stating that the law applies for the fixation of CO_2 by plants, and that transpiration is a part of this process and would therefore follow the same law.

Warburg[11] gives a diagram showing the relation of temperature to the respiratory rate of Chlorella, from which Table 5 is prepared.

Table 5. — Respiratory Rate of Chlorella

Temperature		Respiration	Q_{10}	A
°C.	°F.			
0	–	–	–	–
10	50	37		
			1.89	10,500
20	68	70		
			2.07	12,800
30	86	145		
Mean		–	1.98	11,600

These results appear to confirm Meyer's conclusion that transpiration follows Van't Hoff's law with a Q_{10} of approximately 2 ($A=11,800$).

The Frequency of Chirping of Crickets

The male of the common snowy tree cricket (*Oecanthus niveus*) produces a shrilling sound or chirp, made by raising the upper or fore wings and rubbing them on the hind wings. The noise is due to the clear drumhead-like area in the middle of the fore wings, forming a resonant surface; on the hind wings is a raised toothed ridge, which rubs on the drumhead above it. It is noticeable that the chirps of the cricket increase in rapidity as temperature increases, also that crickets tend to chirp in unison.

The speaker has for some time made observations of cricket chirps and temperature, and the results of some of these are plotted on Fig. 2.

While a few of these observations plot discordantly, on the whole they are consistent, and enable the establishment of an approximate curve of relation between chirps and temperature.

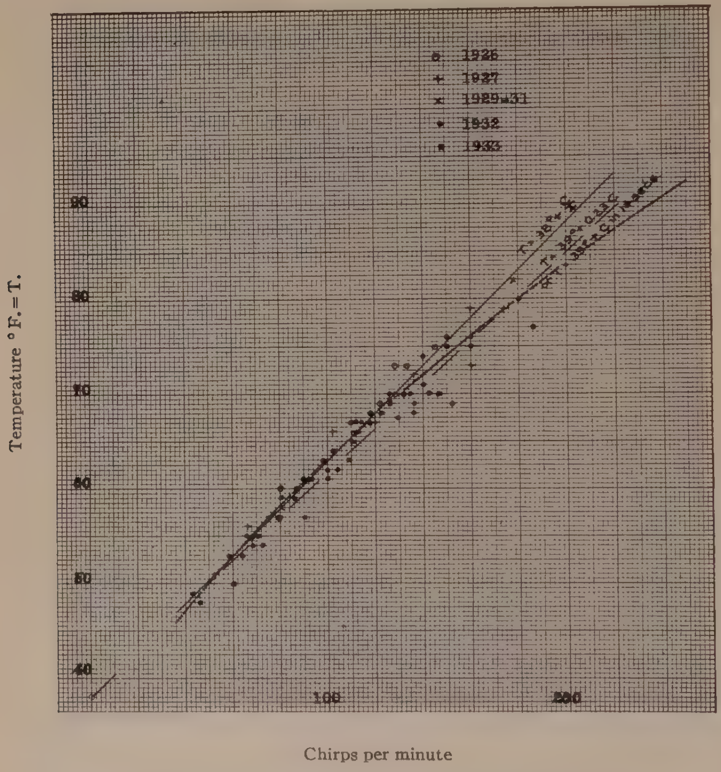

Fig. 2. — Relation of Cricket Chirps and Temperature Observations by H. K. Barrows

An approximate straight line relation is —

$$T=38°+\frac{C}{4},$$

where T = temp. °F.
and C = chirps per minute.

The formula $T=39°+0.23\ C$ is slightly better in fitting the upper part of the observations.

Any straight line relation is approximate, however, and the curve shown on Fig. 2 is more accurate and has been used in obtaining the results in Table 6.

Table 6. — Chirps of Crickets

Temperature		Cricket Chirps per Minute	Q_{10}	A
°C.	°F.			
10	50	44		
			2.72	16,500
20	68	120		
			1.76	10,000
30	86	212		
Mean		–	2.24	13,200

The Frequency of Shrilling of Locusts

Mr. E. J. Lynde of the Pasadena, California, water department has made observations of the shrilling of locusts as affected by temperature. Locusts and grasshoppers, of the same order (*Orthoptera*) as crickets, have their sound organs at the inner base of the fore wings and produce their shrilling sound by rubbing thighs against the fore wings. Lynde's observations, numbering fifteen or twenty, were made in 1935 and were all made with one specimen. He developed an approximate straight line curve relation of

$$T = 50 + 0.26\ C$$

(with notation as previously given for crickets). This would give the results in Table 7.

TABLE 7.—SHRILLING OF LOCUSTS

TEMPERATURE		Shrilling of Locust	Q_{10}	A
°C.	°F.			
10	50	0	–	–
20	68	69	–	–
			2.0	12,200
30	86	138		
			1.5	7,700
40	104	208		
Mean		–	1.75	10,000

More extended data would no doubt indicate that here, as with cricket chirps, a curved rather than a straight line relation would fit the upper portion of the observations better. It is likely that $Q_{10}=2$ also applies fairly well in this case.

RHYTHMIC ACTIVITIES OF ARTHROPODS

Crozier and Stier[12] have listed values of A as follows for rhythmic activities of arthropods:

ACTIVITY	A
Parajulus, velocity of progression	12,250
Ants, velocity of progression	12,250
Tree crickets, frequency of chirping (Shull)	12,200
Fireflies, frequency of flashing	12,200
Limulus, frequency of heartbeat	12,000
Ceriodaphnia, frequency of heartbeat	12,100
Bombyx larvae, frequency of heartbeat	12,200
Added by Hoagland:	
Tent caterpillar, frequency of pedal movements	12,200
Cockroach, frequency of heartbeat	12,500

These all show remarkable uniformity in A at about 12,200 (Q_{10} = about 2.1).

Digestion Rate of Sludge in Sewage Treatment

In the treatment of sewage by sedimentation and separate sludge digestion the sludge is pumped into a tank or digestion chamber, with a resulting anaërobic decomposition of solids. To expedite digestion (1) the incoming solids are thoroughly mixed with digesting sludge, and pocketing of fresh solids thus avoided; (2) the settled solids are introduced while fresh, *i.e.*, before acid fermentation has set in; and (3) the digestion tank is maintained as warm as practicable.[13]

Temperature is therefore an important element in separate sludge digestion treatment, and its effect in stimulating digestion and lessening the necessary time of sludge storage has been given considerable attention.

Fair and Moore[14] use the reciprocal of the relative time of digestion as a measure of the overall rate of digestion and to indicate reaction velocity. They have for the results of various observers plotted temperature °C. against relative rate of digestion, using the rate at 25° C. as unity, and from their plot[15] the results in Table 8 are taken.

Table 8.—Relative Digestion Rate of Sludge

Temperature		Relative Digestion Rate	Q_{10}	*A*
°C.	°F.			
10	50	0.36		
			2.00	11,400
20	68	0.72		
			1.78	9,600
25	77	1.00		
28	82	1.20		
to			0.0	0
42	108	1.20		
42	108	1.20		
			1.67	10,000
52	126	2.00		
			2.09	14,500
55	132	2.50		

As will be noted from Table 8, from 10° to 25° C. Van't Hoff's law applies and Q_{10} is in the vicinity of 2; this is again true between 42° and 55° C., but from 28° to 42° C. the curve is entirely at variance, with little or no increase in activity shown. Fair and Moore suggest that temperatures between 28° and 42° C. are progressively more unfavorable to the existence of the non-thermophilic organisms which do the work of digestion below 28°. Thus any increase in the velocity of the chemical reactions by which the bacteria sustain life and convert solids into gas and residue is compensated and ultimately turned into a net decrease in activity by the adverse effect of the higher temperatures on the bacteria themselves. In practical sludge digestion little is apparently gained by raising the temperature above 28° C. (82° F.), and nothing by raising it above 33° C. (91° F.) unless intending to go all the way to 50°–55° C. in order to take advantage of thermophilic digestion. It is to be noted that this is not true of activated sludge, which appears to be benefited continuously up to 55° C.

OXYGEN DEMAND OF POLLUTED WATERS

The decomposition and mineralization of organic matter by bacterial activity are accompanied by depletion of the dissolved oxygen. The demand of sewage for oxygen therefore reflects both the nature and quantity of the organic matter it contains. The biochemical oxygen demand, or the rate of aerobic decomposition of sewage, is a quantitative measure of the pollutional characteristics of a river water, sewage or industrial waste.

The variation in oxygen demand with temperature has been studied by Pleissner, Streeter and Phelps[16], from whose investigations Table 9 is prepared, using average values of the results obtained.

TABLE 9.—OXYGEN DEMAND OF POLLUTED WATERS

Temperature		Two-Day Oxygen Demand Values	O_{10}	A
°C.	°F.			
0	32	32	2.00	10,500
10	50	64	1.56	7,350
20	68	100	1.40	6,000
30	86	140	1.30	5,000
40	104	182		
Mean		—	1.56	7,500

For the ordinary range of river temperatures, 0°–20° C., Q_{10} would average about 1.8 ($A = 9{,}500$), or not greatly at variance with Van't Hoff's law; for higher temperatures Q_{10} falls off, reaching 1.3 for 30°–40° C. and averaging for the entire range, 0°–40° C., 1.56 ($A = 7{,}500$).

RATE OF GROWTH OF ICE SHEET

The rate of growth of ice sheet in still water at different temperatures has been studied both experimentally and in theory. Barnes[17] gives a table based upon an analysis of actual observed data which he has found to hold quite accurately for fairly large temperature differences, and from which Table 10 has been prepared.

TABLE 10.—RATE OF GROWTH OF ICE SHEET

THICKNESS OF ICE (INCHES)	TEMPERATURE						
	−10° C. (14° F.)	−20° C. (−4° F.)		−30° C. (−22° F.)		−40° C. (−40° F.)	
	Time (Days)	Time (Days)	Q_{10}	Time (Days)	Q_{10}	Time (Days)	Q_{10}
1	.089	.043	2.03	.028	1.53	.022	1.27
6	1.95	0.98	2.00	0.65	1.50	0.49	1.32
10	5.19	2.60	2.00	1.73	1.50	1.29	1.34
12	7.38	3.69	2.00	2.46	1.50	1.85	1.33
24	28.60	14.30	2.00	9.53	1.50	7.15	1.33
36	63.69	31.85	2.00	21.23	1.50	15.92	1.33
Mean			2.00		1.50		1.33
A			11,400		6,850		4,700

In the case of ice formation the rate of growth, of course, varies *inversely* with temperature, but at ordinary winter temperatures in this latitude (−10° C. to −20° C. or 14° F. to −4° F.) appears to have a Q_{10} of -2.0 ($A = -11{,}400$). For low temperatures Q_{10} and A diminish, becoming -1.33 and $-4{,}700$, respectively, for the temperature interval −30° to −40° C. (−22° to −40° F.).

BIOLOGICAL STUDIES OF TEMPERATURE EFFECTS

Many biological studies have been made following the temperature method of analysis, and the results of some of these are listed in Table 11.

TABLE 11.— BIOLOGICAL STUDIES OF TEMPERATURE EFFECTS

No.	PROCESS	Reference	Q_{10} (Approximately)	A
1	Frequency of heart rhythm of silk worm .	18	2.10	12,250
2	Frequency of breathing movements of gold fish	19	2.70	16,500
3	Frequency of heartbeat in *Nereis virens Sars*	20	1.65 to 4.4	8,300 to 24,400
4	Respiration rate of earthworm . . .	21	1.95	11,000
5	Rate of breathing movements of frog .	22	1.70	8,600
6	Rate of breathing rhythm of fondulus —	23		
	In salt water		1.67	8,400
	In fresh water:			
	Below 9.5° C.		2.72	16,400
	Above 9.5° C.		2.00	11,300
7	Oxygen consumption of earthworms . .	24	1.95	11,040
8	Human brain waves, frequency of alpha rhythms:	25		
	Normal		1.63	8,000
	Paretic		1.96	11,000
	More paretic		2.62	16,000
	Mean	–	2.14	12,500

One of the most interesting of the biological studies in Table 11 is that relating to the frequency of alpha rhythms in human brain waves as described by Hoagland:[25]

> Recent work from several laboratories has shown that various groups of cells in the mammalian brain exhibit what appears to be "spontaneous" electrical activity, providing rhythmic waves of potential variations at characteristic frequencies. The most readily detectable of these rhythms, the alpha rhythm, also called the "Berger rhythm," has been shown to arise in a localized region of the occipital cortex. In man these occipital potential fluctuations of from 50 to 100 microvolts may be ob-

tained, with a suitable recording system, directly through the skull merely by placing pad electrodes on the head, one preferably on the occiput, and the other on a region anterior to the vertex. . . . The alpha waves occur at a frequency of about 9 or 10 per second. The rhythm is remarkably constant in a given individual, and seems to be characteristic of activity of the otherwise "resting" cortical cells, since pattern vision usually inhibits it, probably due to a breaking up of the synchronized beating of the cells, when they are thrown into specific patterns of activity by light. The rhythm is also inhibited during emotional disturbances.

It is reasonable to suppose that the alpha rhythm from the occipital cortex may be due to the activity of a "relaxation oscillator" system resulting from continuous, entirely *non-rhythmic*, metabolic events going on in the cortical cells. The frequency of the rhythm might, therefore, be expected to be directly proportional to the speed of these local metabolic events.

(Relaxation oscillations, in contrast to those produced by the reactions between elasticity and inertia, for example, are rhythms set up from any system in which a continuous source of energy stores up a potential, which at some cortical value discharges itself and in so doing tends to channel its pathway, thus releasing the charged store.)

The experiments to examine the alpha rhythm as a function of temperature were made by Hoagland upon ten subjects at the Worcester State Hospital.

The subject was placed in a bed, thoroughly wrapped to prevent heat loss, and his temperature elevated by passing high frequency alternating currents through his body. Rectal temperatures were taken every fifteen minutes with a clinical thermometer during the one and a half to two hours that were necessary to elevate the temperature. Immediately after recording each temperature, in the course of the procedure, electrodes were put on the subject's head, the high frequency current turned off, the subject blindfolded, and one or two minutes later the alpha rhythm was recorded continuously for some fifty seconds by a Larceau recording system, consisting of an amplifier and ink-writing undulator recording on a paper tape.

Of the ten subjects studied, three were normal, one a mentally normal multiple sclerosis patient, and six, general paretics in various stages of the disease.

It is of interest to note from Table 11 that the normal subjects showed an *A* of 8,000, the mildly paretic 11,000 and the more advanced paretics 16,000, suggesting, according to Hoagland, "a definite shift in the chemical pacemaker determining the oscillations with the more advanced state of the disease."

SUMMARY

In Table 12 is given a summary of values of Q_{10} and A prepared from the eleven different groups of processes or reactions, noting that Group 11 shows the results of eight different studies and that twenty-four experimental items in all are included.

TABLE 12.— TEMPERATURE EFFECT STUDY

Group	PROCESS OR REACTION AS AFFECTED BY TEMPERATURE	General Nature	Q_{10} (Approximately)	A
1	Variation in pressure of saturated vapor	Physical		
	Water		1.87	10,600
	Carbon bisulphide		1.48	6,700
	Ethyl alcohol		1.82	10,200
2	Evaporation from water area	Physical	1.75±	9,000±
3	Photosynthesis of plants:			
	Blackman reaction	Chemical	2.0	11,800
	Low light intensity	Photochemical	1.	0
4	Transpiration of Chlorella	Physical	1.98	11,600
5	Frequency of cricket chirps	Physiological	2.24	13,200
6	Frequency of locust shrills	Physiological	1.75	10,000
7	Rhythmic activities of arthropods	Physiological	2.1	12,200
8	Digestion rate of sludge (except 28° to 42° C.)	Biochemical	1.67 to 2.09	10,000 to 14,500
9	Oxygen demand of polluted water	Biochemical	1.56	7,500
10	Rate of growth of ice sheet	Physical	−2. to −1.33	−11,400 to −4,700
11	Miscellaneous biological studies	Biological		
	Range		1.67 to 4.4	8,400 to 24,300
	Mean		2.15	12,500

The general nature of each process is also stated, and in accordance with this approximate group results are given in Table 13.

TABLE 13. — GROUP SUMMARIES OF TEMPERATURE EFFECT STUDIES

Number of Group Items	NATURE	Q_{10}			A		
		Maximum	Minimum	Mean	Maximum	Minimum	Mean
8	Physical	2.54	1.33	1.85	16,000	4,700	10,000
2	Chemical	2.6	1.0	1.9	15,700	0	10,200
3	Physiological	2.16	1.75	2.0	12,600	9,200	11,400
3	Biochemical	2.09	1.56	1.8	14,500	7,500	10,700
8	Biological	4.4	1.67	2.15	24,300	8,400	12,500
	Mean of 24 items	2.65	1.46	1.94	16,600	6,000	11,000

Q_{10} has a mean value of 1.94, a mean maximum of 2.65, and a mean minimum of 1.46.

A has a mean value of 11,000, a mean maximum of 16,600, and a mean minimum of 6,000.

It will be also noted from Table 13 that the various groups show mean values of Q_{10} and A which vary through a relatively small range. Thus the mean Q_{10} varies from 1.8 to 2.15, and A from 10,000 to 12,500.

CONCLUSION

In this study a wide variety of processes and reactions have been included, and at first thought it might seem unlikely that such diverse processes and reactions would show any degree of uniformity in the temperature characteristics Q_{10} and A in even their comparison by averages. They are all, however, fundamentally chemical reactions in that they involve molecular activity.

Thus, variation in the pressure of saturated water or other vapor with temperature involves molecular activity, although grouped as a physical process. The same is true of evaporation from water area and also the formation of sheet ice.

Photosynthesis involves chemical reactions, stimulated by light in the presence of chlorophyll. Transpiration is a part of the process. Again molecular activity is involved.

The chirps of crickets, the shrilling of locusts and the rhythmic activities of arthropods, as well as the numerous processes included under the other biological studies in Table 11, are all phases of molecular activity in connection with the metabolism of these creatures.

Sludge digestion and oxygen demand of polluted water involve bio-chemical processes and again molecular activity.

It is entirely consistent, therefore, that these various processes and reactions should show a degree of harmony in respect to their temperature reaction characteristics, Q_{10} and A, and this appears to be the case.

The study of the temperature reaction coefficients Q_{10} and A is likely to be of value in the field of hydrology, particularly in studies of run-off based upon the determination of water losses as compared to precipitation. This has already been realized and applied in the determination of transpiration losses, but requires more experimental studies and research. Evaporation from water areas may perhaps be studied from this angle, particularly in the recent so-called rational or solar energy method of its determination, which is in a formative stage of development, requiring more research and experimentation. Soil evaporation is another type of water loss of great importance, in which the study of the temperature coefficients may be useful.

While little new basic data have been contributed by this study, it is hoped that the review of a rather broad field of processes and activities as related to temperature reactions may be helpful in a liason of the fields of chemistry, biology and engineering.

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STRUCTURAL STEEL BEARING PILES — THEIR USE AND CAPACITY

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(Presented at a meeting of the Designers Section of the Boston Society of Civil Engineers held on January 13, 1937)

While structural steel bearing piles have been in use in this country for over thirty-five years, it is only during the last four or five years that their use has become at all widespread. Their advantages and possible methods of use are just beginning to be exploited by the steel companies, and adequate data regarding their use and capacity are so meagre that many engineers have scarcely even heard of them. Even those who know most about them are not fully aware of the best details of their employment, and hesitate to take full advantage of their capacity, which is admittedly a difficult quantity to determine except by actual test. Their use has been approved by most of the state highway departments and Federal agencies, and is provided for in the new building code for the city of New York.

Metal piles have been used for nearly one hundred years, since 1838, in the form of cast-iron pipes and solid wrought-iron shafts, principally in the construction of ocean piers and wharves. Such piles had little penetration and depended for their supporting power on either a disk or a broad screw flange attached to the foot. The disk pile was jetted down a few feet, just enough to get below the line of possible scour, while the screw pile was turned into place, sometimes with the aid of a jet also. It is doubtful if there are any such structures still in use in this country, but there are a number of them in Central America, built mostly by European firms. Some of them have been in active use for over thirty years. There are two in Costa Rica, one at Puenta Arenas, on the west coast, and the other at Puerto Limón, on the east coast.

The latter is a T-shaped structure, designed and built nearly fifty years ago by a Belgian firm for the Costa Rica Railroad. The approach, or stem of the T, is about 1,200 feet long and 30 feet wide to accommodate two railroad tracks. The cross of the T is 1,000 feet long and 75 feet

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wide. The piles are of two types, the outside line around the entire wharf being a solid wrought-iron shaft 6 inches in diameter. All interior piles are cast-iron cylinders 18 inches in outside diameter, and with walls 1¼ inches thick. They stand in 30 to 35 feet of water at the outer end of the pier and are completely braced down to the ocean bottom with three or four panels of pin-connected rod bracing, with turnbuckles for adjustment, and double channel struts. The piles are spaced about 15 feet apart in the bents, with 20 to 24 feet between bents.

The solid wrought-iron piles show appreciable damage from corrosion, particularly at the water line. The cast-iron piles show no signs of any deterioration whatever. The original bracing rusted through and was replaced about fifteen years ago, an expensive operation requiring four years' time. All piles are fitted with screw ends, and a number of extra tips and sections of piling are still stored near the site.

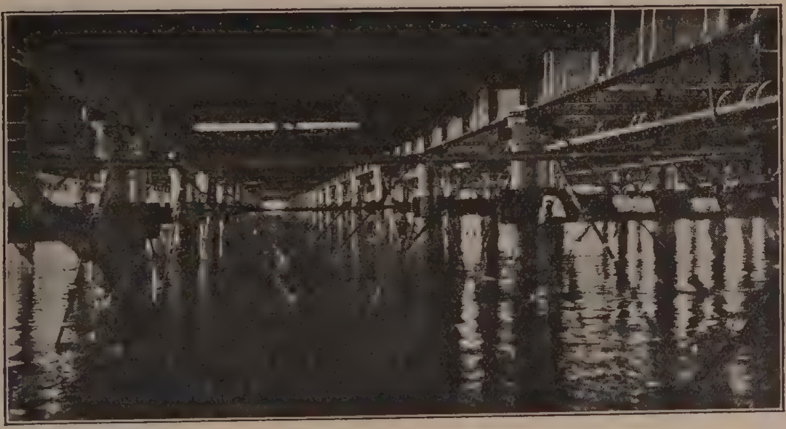


Fig. 1. — The Underside of the Metallic Pier at Puerto Limon. Note the Heavy Growth of Barnacles; also the Method of connecting the Bracing to the Cast-iron Pipe Piles

The deck framing consists of built-up girders and rolled beams, presumably of wrought iron, and nearly all of the original construction. The deck is of timber plank. The condition of the floor metal is quite remarkable, even the rivet heads in the under side of some of the girders showing no signs of pitting, although only 6 to 8 feet above the water. The metal has been protected with what is known as "Panama Mixture" or "Cunningham's Mixture," consisting of cement, kerosene and coal tar, applied hot.

Considerable emphasis has been given to this all-metal pier, because, in spite of the fact that such construction was in use before the turn of the century, most of our piers in this country today still are founded upon timber piles, which are more susceptible to damage and deterioration, and have much less capacity. It is true that steel pipes and tubes filled with concrete have been used as piles in building foundations on rock, particularly in New York City and vicinity, for about thirty-eight years, but most of our harbor and dock work has continued the use of timber piling. Concrete piles and framing have been used in pier construction to a considerable extent, but concrete is not entirely satisfactory and has its limitations. The most flexible type of material and construction available for this purpose is certainly steel, but it is only in the past few years that there has been any real tendency in this direction. The recently reconstructed platforms of Commonwealth Pier No. 5 at South Boston is an excellent example of such an all-steel structure, and is a credit to those responsible for its ingenious design and construction.

The use of structural steel bearing piles in this country originated shortly before 1900, in highway bridges in Nebraska and adjoining States. The earliest piling, in the form of I-beams and fabricated sections made from plates and angles, was used in what proved to be a successful effort to meet the serious problem caused by the undermining of bridge piers and abutments on pile foundations in heavy gravel and sand, which limited the depth to which any other type of piling could be driven. It was demonstrated that the steel piles could readily be driven to depths safely below the scouring action of the streams. The steel piles also solved another serious problem of providing the necessary structural resistance to damage from impact of ice floes.

A typical instance of such a condition was the difficulty of maintaining safe bridge crossings over the Platte River in Nebraska. This river is wide but shallow and has an extensive drainage area. Because of these characteristics, ice floes developed which were out of all proportion to the discharge capacity of the stream. At such times, the rising river would open up new channels with a terrific scouring action, in many cases undermining the bridge structures. The only salvation for them was the development of foundation piles capable of being driven well below the level of possible scour. I-beams and fabricated steel piles provided that solution.

After 1908, when the rolled steel H-column section was first introduced in this country by the Bethlehem Steel Company, the H-section was soon recognized as particularly well suited for use as piling. So successful was the H-piling that the use of the older types of structural steel piles was soon abandoned in favor of this section.

The early use of steel H-piles, primarily as a means of combating stream scour and resisting attacks from floating ice, also demonstrated that greater loads than usual could be safely carried by the more deeply driven specimens. A simple method of testing the bearing capacity by means of hydraulic jacks was developed, and a great many loading tests were made which demonstrated the high bearing capacity of the steel H-pile. The greater loads that could be placed on them thus permitted their use with initial economy in addition to providing long-term economy by affording security against scour and ice, and other damage.

Through recognition of its many inherent advantages this type of piling was widely adopted throughout the West. Even in 1932, over 10,000 bridges stood on steel H-piling just in the territory between the Continental Divide and the Mississippi River. In this area there is a great diversity of soil and climatic conditions. Within recent years there has been a great increase in the use of steel H-piles in other parts of the country which can be attributed largely to the excellent service rendered by the early installations of thirty to thirty-five years ago. Generally it has been shown that the loss of metal from corrosion was too small to be of any consequence. During the last few years the use of steel H-piles has extended to practically every class of work for which bearing piles are required, and particularly where high bearing values can be developed.

One of the principal uses of steel H-piling has been for highway bridges of the trestle-bent type for which it is particularly well adapted. Bents of steel H-piling can be easily and adequately braced and capped. Steel H-piles are also being used extensively under concrete piers and abutments of bridges in place of timber piles. There is now hardly a State in this country without some examples of highway bridges founded on steel H-piling.

Several prominent railroads are using steel H-piling for permanent and temporary bridges and trestles. Some of these temporary trestles employ only two steel piles in each bent, where formerly five timber piles were used. Loads up to 70 tons are common. The high salvage value of the steel piles is an important consideration in their use in temporary structures, since they are not nearly as susceptible to damage in driving and pulling as are timber or concrete piles.

In the construction of ocean piers and wharves of the platform type, steel H-piles are being used to advantage, particularly where the waters are infested by marine borers.

Other types of construction in which H-piles have been used as the foundation are locks and dams, industrial buildings, grain elevators, water tanks and falsework for bridge erection. The new Civic Center Building in San Diego, California, now under construction, is a monumental building founded on 1,500 steel H-piles amounting to 1,300 tons. The necessity for designing against earthquake shock was a determining reason for their use in this structure.

Steel H-piles are, of course, utilized to greatest advantage in locations where they can be driven into hard ground material, such as hardpan or shale, or to firm bearing on solid rock. In such cases extremely high bearing capacities can be developed; in fact, the capacity of the pile as a column or at some maximum allowable unit stress. In cases where an extreme penetration is necessary to reach a firm stratum, it can be accomplished readily with steel H-piling. On account of the facility of splicing, even to develop 100 per cent of the pile section, there is practically no limit to the length which can be used.

While similar results might be shown for other types of piles if tested so extensively, the many loading tests to failure which have been made on steel H-piles when used as friction piles have generally shown that much greater loads could safely be applied than were indicated by an ordinary pile-driving formula of the Engineering-News type. The steel H-pile, on account of its material and shape, transmits a higher proportion of the driving energy into effective work of soil penetration than do piles of other material. For this reason, it is obvious that with a given penetration during the driving, a steel H-pile overcomes more frictional resistance than does another type of pile which cushions the hammer blow to a greater extent.

Consideration should be given to the life of steel piling. When exposed and unprotected, it is, of course, subject to corrosion, but evidence shows that the actual amount of deterioration from this cause is generally over-estimated. In most structures where steel piling is used, only a small portion of it requires special consideration from the standpoint of corrosion. If any portion is to be submerged, it can be protected by proper paint application before installation. The portion extending above water or ground level can be protected either by painting or by encasement in concrete. As to the best coating for steel piling subjected to sea water exposure, although there are differences of opinion, experience indicates that a coating of tar or of asphalt emulsion seems to offer one of the best protections. Probably a priming coat of good red lead paint is desirable, and some of the newer paints having a synthetic vehicle base have proved extremely suitable for the purpose.

Most concern as to the life of steel bearing piles is usually felt for that part driven into the ground. Tests and inspections of steel driven into the ground indicate that corrosion decreases with the depth as free oxygen is excluded from contact with the steel and as the earth becomes more compact with its depth. The products of initial corrosion accumulate on the surface of the steel and form a protective coating which is not removed as is rust on unburied steel. This accounts for the continuous decrease in the rate of corrosion loss of buried steel. The Bureau of Standards' investigation of corrosion of ferrous pipe materials buried in shallow trenches in forty-seven different soils in the United States indicates, after a twelve-year study, an average corrosion penetration of the open hearth steel specimens amounting to .00084 inch per year. At that rate of corrosion loss, it would require about eighty years for the loss of the outer $\frac{1}{16}$ inch of metal. Most specifications for the use of steel pipe piles provide that in calculating the effective area of the steel, the outer $\frac{1}{16}$ inch be deducted from the cross section of the pipe to provide for corrosion loss. Apparently, however, no pipe piles which have been examined have shown more than about $\frac{1}{64}$-inch loss in twenty-five years.

There is now sufficient information available on the life of steel piles to indicate that the structures which they support will be outmoded in most cases long before concern will be felt for the life of the piles. Exceptions are piles exposed to abrasive action of sand in strong currents of water, to an unusual form of growth found in certain tropical waters, and to certain unusual soil conditions. In such cases special methods of protection are necessary.

Just recently some steel sheet piling was removed from the municipal docks in Miami, Florida. This piling had been installed in bulkhead construction for ten years. The portion of the pile which had been below harbor bottom appeared to be just as smooth and in as good condition as when it was new. The portion between harbor bottom and low-tide level was practically in the same condition. Between high and low tide levels, the pile was only slightly rusted. The roll mark was still clearly visible. This slight corrosion between tide levels was due to the protection afforded by a coating of oil from the oil film ever present on most harbor waters. The worst rusting was above high-tide level. The pile was rusted and pitted in some places on the water side only, but not badly. All of the piling pulled was in such good condition that it was driven in another location, end for end, without cutting off any of the top portion.

It is understood that on an important viaduct structure in the

South, to be built with P. W. A. funds, a thirty-year loan was granted for the structure on steel H-piling or precast concrete piles, while on a similar structure built a few years ago on creosoted timber piles only a fifteen-year loan was granted.

Probably the most frequent and troublesome question raised regarding steel piles — and any other kind of piling, for that matter — is, what load will they carry, and how far must they be driven. Steel is a more expensive material for piling than either timber or concrete, and in order to fully justify its use economically it must carry greater loads, proportionately, than other types of piles.

Steel piles are usually driven to place by a succession of impacts, and theoretically it is quite possible to derive a mathematical expression for the bearing capacity of a pile so driven, given the necessary physical constants and the driving data. Such formulas in their complete form are so complicated as to be impracticable for everyday use, and even in their modified form they have not yet come to be generally accepted, although they can be made to give fairly satisfactory results as compared to actual tests. A comparison of formula results with actual tests will be given later.

A brief discussion of pile-driving phenomena may not be out of place here, preparatory to a résumé of some extensive load-bearing tests of steel H-piles made recently by the Bethlehem Steel Company on the West Coast. Experience has shown that values furnished by pile-driving formulas often have nothing in common with those determined by loading tests, and may be either too small or too large, depending on the various factors involved. The reason for this is not so much a defect in the formula, if it be of a rational type, as the fact that the forces which resist the penetration of the pile under impact are not those which resist the penetration under static load. It is well known that the bearing capacity of a pile depends upon two factors: (1) the point resistance, and (2) the frictional resistance acting along the sides of the pile. If these two resistances depended only on the character of the ground, then the resistance of the pile against driving and its static bearing capacity would be identical. However, it can be shown that either one of them may be very different according to whether the pile is forced down slowly or driven down by impact.

For this purpose soils are classified according to two main types, the first consisting of ordinary sand, gravel and permeable granular soils, and the second of very fine grained silts, soft clays, and the like. The resistance of piles driven into the first class of materials (sand and gravel) is practically the same while being driven as it is under a static load, and for this case any rational pile-driving formula, even in its simplified form, will give reasonably reliable results.

On the other hand, the resistance to piles driven into the second class of materials (silts and soft clays) is of two kinds, — one, the dynamic resistance, as to driving, consisting of a large point resistance and a negligible frictional resistance; the other, the static resistance after driving has ceased for some time, consisting of the full frictional resistance, which becomes quite large, and a small point resistance. Obviously, no pile formula can take account of any but the dynamic resistance, hence the difficulty of obtaining reliable bearing values by means of a formula in this kind of soil. The most reliable means of determining the capacity of any pile in this kind of soil is by actual test, plotting a load-settlement curve, and basing the working load on some definite settlement or slope on this curve.

It has been found generally that pile-driving formulas give results which are too low when compared with tests for piles that are driven in clay soils. Every engineer who has driven piles in soils of which clay forms any appreciable part is familiar with the sickening ease with which such piles may often be driven, but is surprised at their comparatively great resistance to static loading, particularly after a period of rest. The reason is generally accepted to be the fact that during driving water is squeezed out of the material and tends to lubricate the pile. After a period of rest this water is reabsorbed and the soil settles firmly around the pile, developing its full frictional resistance.

Up to the present time one of the most useful means of employing steel H-piles has been in locations where, although rock is available, it is at great depth or overlaid with one or more layers of hard material in beds too thin or unstable because of the underlying softer strata to make good bearing mediums in themselves. Steel H-piles are well adapted to hard driving which would shatter other materials, thus making it possible to penetrate such hard strata, even of small boulders and hardpan, and to reach solid rock. Where the penetration is great they are easily spliced and lengths of over 100 feet for such piles are not uncommon.

The greatest problem in utilizing not only steel H-piles but every kind of pile is to increase their bearing capacity in the clays and soft wet soils, and various means have been proposed. Timber and concrete piles are at a disadvantage with steel H-piles when it comes to hard driving through sand and gravel soils, but they have an advantage in the soft soils in that their bulk displaces and tends to compact the adjacent soil and thus increase its stability and bearing qualities. Steel piles are easily adapted to this kind of soil, however.

Originally short scabs consisting of pieces of steel H-sections were riveted or welded to the flanges of the pile, usually near the point, sometimes higher up, and in several instances they have proved effective. Another means employed has been the use of horizontal plates used in pairs between the flanges on opposite sides of the web, and welded to both flanges and to the web. These are known as "core-stoppers" and may be located at various points on the embedded length of the pile. They are simple to apply and their use has also proved effective in some cases, so much so that an elaborate series of bearing tests based on their use was conducted recently and will be described in detail presently.

Probably the most effective means of transforming the slender, streamlined H-pile into a bulky displacement pile is by means of lagging. This generally takes the form of pairs of timbers bolted either to the web or to the flanges of the H-section, or to both, for the full length of penetration. A steel bracket, welded or riveted to the shaft, is usually employed for the upper end of the lagging timbers to bear against. This form of lagging was employed on most of the 192 bearing piles furnished for the United Fruit Company's pier for the government of Panama, built in 1935. Another form of lagging is the use of steel plates welded to the flanges and completely boxing in the H-section for any desired length. Piles lagged by some means, and driven deeply enough, can usually be made to develop the bearing value necessary to justify their use economically.

It is, of course, impossible to give definite values which steel H-piles can carry for any and all conditions. Where they are driven to rock and supported by reasonably firm soil throughout their length they will easily develop their capacity at the maximum allowable unit stress for a short column, say, 15,000 pounds per square inch, or about 75 tons for the lightest 8-inch section, and 150 tons for the 12-inch H. The fact that the bearing on the rock is not uniform over the area of the pile, and even though the pile may have been damaged in an endeavor to seat it in the rock, has been shown by test not to be detrimental either to the pile or to the rock as regards its ability to safely carry maximum loads. Several tests are on record where H-piles have carried loads corresponding to the elastic limit of the steel without any damage or appreciable settlement.

Capacity loads as a column, fully supported or otherwise, are also on record for steel H-piles driven into sand and gravel soils. Hard, compacted beach sand, for example, is extremely difficult to drive through, and sometimes jetting must be resorted to in order to get the required penetration for lateral stability. Such piles easily develop their capacity as a column, assuming that they are not one of the extremely heavy H-sections. For such soils, as previously mentioned, the bearing resistance may be safely estimated by means of a rational type of pile-driving formula and the use of as low a factor of safety as 2. Indicated safe loads of from 50 tons for an 8-inch H up to 100 tons for a 12-inch section are not unusual in good sand and gravel soils.

Clays, silts and soft, wet soils are much more commonly encountered in foundation problems than sand and gravel, however, and it is here the question of a safe load is most difficult of determination. In spite of the writers' firm belief that eventually it may be possible to use a rational type of pile-driving formula with a factor of safety as low as 2 or 3, it must be admitted that at the present time a loading test is the safest method of determining the capacity of an H-pile in such soils. On the other hand, most tests which have been made and reported are not sufficiently precise and scientific enough to determine the true capacity of the pile or the factor of safety of the test load with respect to the load which would cause yielding. As a matter of fact, there is as yet no definite or accepted criterion for determining the maximum or the yield load of a pile under test. The most generally accepted criterion is that of no continued settlement under one and one half times the design load and a maximum resultant settlement of not over one hundredth of an inch per ton of load after the load is released.

FIG. 2. — GENERAL VIEW OF NEW STEEL PIER BUILT FOR GOVERNMENT OF PANAMA BY CHIRIQUI LAND COMPANY (SUBSIDIARY OF UNITED FRUIT COMPANY) AT PUERTO ARMUELLES, PANAMA

The New York City Building Code, which recently permitted the use of steel H-piles, uses this criterion for a bearing test, but with a maximum design load of 40 tons. When driven to rock there is allowed 40 tons for an 8-inch H, up to 65 tons for a 14-inch section, with a maximum unit stress of 8,500 pounds per square inch regardless of lateral support.

The maximum load actually supported by a steel H-pile in a soil of the second class, of which the writers are aware, is in the United Fruit Company's Panama pier, previously referred to. There the soil was a very fine, sandy, silty, clay mixture, quite soft and completely saturated. These piles were generally of about 14-inch H section, having a penetration of 40 to 45 feet, and lagged with two or four timbers 10 inches by 12 inches or 12 inches by 14 inches. They have been subjected to actual loads of about 65 tons, and have settled a small initial amount of about one half inch only, without any further settlement. The average load to which any kind of piles in such soil are usually subjected is not over 30 tons and often less, depending upon the seriousness of any possible settlement.

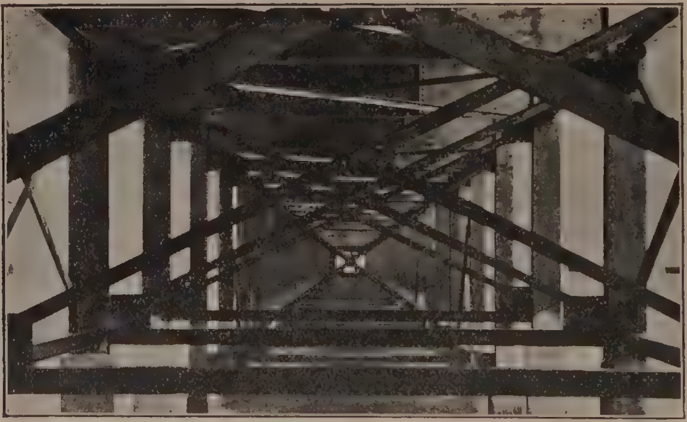


Fig. 3. — Bracing under Approach Trestle, Pier at Puerto Armuelles, Panama

As a general rule, lacking any other data or criterion, it may usually be safely assumed that even in the softest soils which may still be regarded as a soil and not as a liquid mud, the safe bearing capacity can be estimated at 2 to 2½ pounds per square inch of superficial pile area, or about 300 to 350 pounds per square foot of surface. For a 12-

inch H-section this amounts to about a ton per foot of penetration. Even if the plain pile will not develop this load, which is unusual, it can be made to do so by some kind of lagging. For stiffer and dryer clay soils the unit frictional value will increase to as much as 7 or 8 pounds per square inch, or 3 to 3¼ tons per foot for a 12-inch H. Some of this resistance is, of course, due to point resistance and not entirely to friction. Pulling tests having indicated frictional values of 5 and 6 pounds per square inch are to be expected in ordinary clays, and higher values are on record. It is realized that usually the soil between the flanges "balls up" and compacts to such an extent that the friction between it and the pile becomes greater than the cohesion of the soil itself, and this compacted volume of soil becomes, in effect, a part of the pile itself and will pull out of the ground with the pile. In the better soils it is so hard that it must be removed with a pick or even a pneumatic chipper. In spite of this phenomena it is still desirable, and perfectly permissible, to consider the resistance of the pile in terms of its superficial area so long as all test data are interpreted in this manner, as has been assumed in the foregoing values.

There is not much data on the bearing capacity of steel H-piles in groups, but generally their greater individual capacity permits them to be spaced farther apart than other types, and so far as is now known there is not much danger of the group capacity being appreciably less than the sum of the individual capacities. This would not be true in the case of lagged piles, however, and the same limitations would apply to them as to any other displacement pile when driven closely together, say less than 3 feet. No pile can support more than it can be supported, and there is no magic about steel H-piles which gives them any more supporting power than can be derived from the soil or strata which supports them. The compressed core of earth formed in the channel grooves of an H-pile in some types of soils undoubtedly results in a large volume or "bulb" of material near the lower end which helps to spread the load laterally. Where such a condition exists the piles must be spaced far enough apart to avoid overlapping of these compressed areas if the full individual capacity of each pile is to be attained. Not much is known about this subject at this time, and there is much room for study.

For the purpose of determining the characteristics of steel H-piles in clay and clay-sand soils, the Bethlehem Steel Company conducted an elaborate series of vertical load-bearing and pulling tests at the site of the San Francisco–Oakland Bay Bridge, in August, 1935. A total of 26 piles were tested, all of 12-inch, wide-flanged sections that had been used as part of the falsework footings for one of the 504-foot truss spans in the

East Bay crossing. These tests are the subject of a separate and very detailed report which has not as yet been fully completed.

Of the 26 piles involved, 8 had plain shafts and 18 were fitted with horizontal bearing plates between the flanges at various locations, already referred to above as core-stoppers. At that time it was believed that such a device might be extremely effective in increasing the bearing power in clay soils, and unfortunately the tests were conducted entirely with that object in view. Most of the bearing tests were of the accelerated loading type, the load being applied in increments of 8.6 tons at intervals of seven and one half minutes. A series of six sustained loading tests of about fourteen hours' duration was also made as a check on possible continued slow settlement which would not be obvious in an accelerated test. Careful settlement readings were made under each load increment by means of Ames dials reading to one thousandth of an inch, and load-settlement curves plotted for comparison of the different piles and as a basis for obtaining the yield load.

Previous to testing, the piles were all used as falsework footing piles, and as such were subjected to a maximum load of about 30 tons each. They were driven with a McKiernan-Terry No. 11-B-2 hammer at an operating speed estimated to give 19,150 foot-pounds energy per blow. After being used as falsework they were divided into three main groups, A, B and C, of eight piles each. The groups were identical as to the types of piles therein, the only difference being in the driving. Group A piles were driven down about 3 feet just previous to testing in order to simulate newly driven piles. Group B piles were redriven approximately 2 inches, or just enough to get a fair average for a test blow under the hammer. Group C piles were undisturbed before testing and were intended to represent piles which had been allowed a period of rest before loading.

The soil at this site was probably typical of what is generally described as clay and clay-sand mixtures. Such sand as existed was very fine and was not of the coarse variety. Four samples of soil at varying depths were obtained from the piles after they were pulled, and were found to agree closely with the very elaborate boring data furnished by the Bay Bridge engineers. The top 13 feet was a surface "mud" of almost pure, very soft clay. Below this was a layer of "clayey-sand" 20 feet thick, containing about 80 per cent of very fine sand and the remainder clay and silt. Under this was an 18-foot bed of very sandy, silty clay, consisting of 56 per cent clay, 25 per cent silt, and 19 per cent sand. The moisture content varied from approximately 25 to 40 per cent.

The plain piles had a total length of penetration of about 40 feet, so that their points were about 7 or 8 feet into this bed of clay. The core-stoppered piles averaged around 34-foot penetration, so that they just about got through the 20-foot bed of clayey sand underlying the 13 feet of surface mud. The approximate boring profile is shown in Fig. 4.

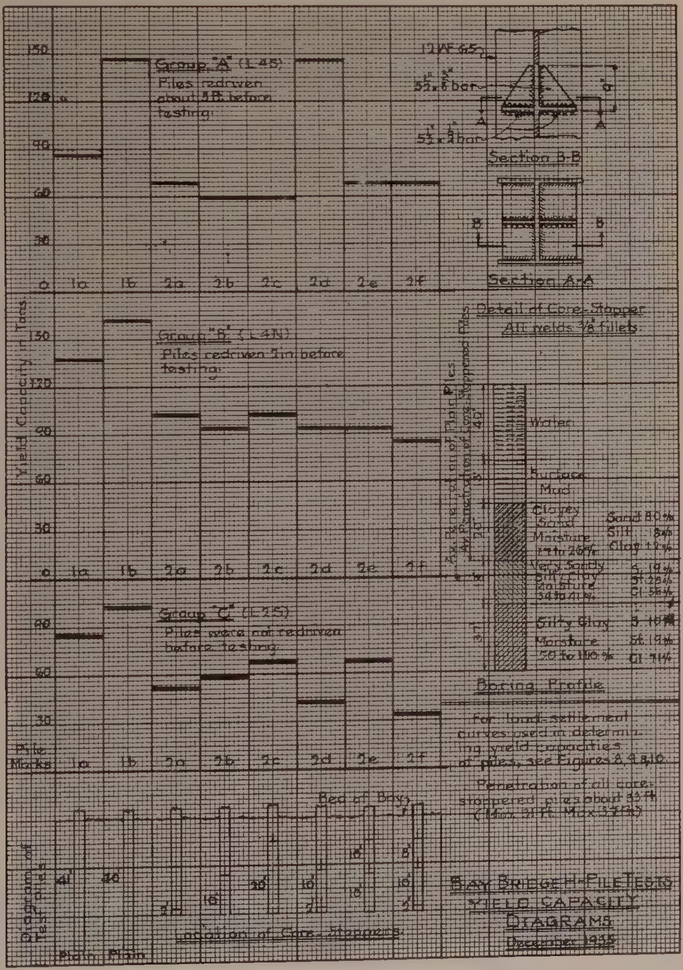


Fig. 4. — Bay Bridge H-Pile Tests. Yield Capacity Diagrams

Each pile tested was located between two others which were used as anchor piles for applying the test load, and was cut off shorter than those two. The test load was applied by means of a 350-ton hydraulic jack, operated by a hand pump, bearing between the top of the test pile and two cross girders supported on the anchor piles. The pressure was measured by a special gauge, purchased especially for test purposes, reading to 100 pounds per square inch and carefully calibrated at the factory.

Pulling of the test piles was accomplished in a similar manner, using two 100-ton jacks placed on top of the cross girders and lifting a short equalizing beam which carried the pulling links which were pinned off to the top of the test pile. Each pile was pulled by applying the jack load in increments of 9.5 tons at four-minute intervals, and Ames dial readings were taken the same as in the bearing test, and pull-yield curves plotted. Pulling was continued until a movement of at least 1½ inches was obtained, and usually 4 or 5 inches.

For the six sustained loading tests a steady load of about two thirds of what was believed to be the yield or failure load was applied and held for about fourteen hours, during which time the Ames dials were read at regular intervals to detect any settlement. After this time the load was increased by the usual increments until a yield load was again reached.

As noted previously there is as yet no definite criterion for determining the yield load or the maximum load which a pile can safely support. As a matter of fact the term "yield load" is not in general use as regards pile loads, and as far as is known it was invented for these tests. As used here the term "ultimate load" refers to the final load applied to the pile regardless of settlement. It is not determined by any criterion and is therefore not very precise, but in general it is the greatest load which could be applied by the means used (hydraulic jacks), which was slow and tedious once the pile had started to really settle. In some cases more load might have been applied, but the load-settlement curves plotted show that the pile had definitely failed by settlement.

The yield load of a pile as here used is, as the name implies, intended to correspond to the yield strength (not the yield point) of any elastic material, such as steel; that is, it is the maximum load which if released would produce only a nominal permanent settlement of the pile, corresponding to a very small permanent set in a tension test specimen of steel, for example. For steel a very common means of determining the yield strength is by the proportional method in which its value is defined as that load producing an increment of strain which clearly exceeds twice the increment of strain taken from the modulus line. In other words, the slope of the stress-strain curve at the yield strength is at least twice that of the straight-line portion of the curve.

Where a series of load-settlement readings are available, such data furnish an easy and yet precise method of determining the yield capacity for piles.

For these tests a slope of the load-settlement curve of four and one half times the slope of the elastic deformation line for the steel alone has been used as a criterion for the yield load of the pile, using only half the imbedded length in figuring the elastic deformation of the piles. This means that the slope of this line for the yield load of the pile is approximately twice that for the yield strength of the steel alone. Since soils are far from being perfectly elastic, and since pile settlements involve both elastic deformation of the steel and the much larger deformation or "give" of the soil, this slope does not seem too great. It gives reasonable results in all cases to which the rule has so far been applied, and always a more conservative result than is prescribed in many building codes in which the maximum safe test load is defined as one which when released shall have produced a settlement of not more than one hundredth of an inch per ton of load.

Some of the conclusions drawn from these tests will now be discussed, which is best done with the help of tables and charts.

Table 1 gives a summary of the pile-driving data. The soft character of the soil is indicated by the fact that all piles penetrated the first 15 to 20 feet under their own weight. The average penetration per blow for the last foot of driving is not truly comparable between the plain and core-stoppered piles, since the total penetration is different; but in spite of this it is obvious that the difference is due to more than can be accounted for by the extra length of penetration or the slightly different character of the soil. The plain piles show a very much smaller penetration per blow, with one exception, C1a. The records show this to have been true for the entire depth and not merely the last foot of driving.

Table 2 gives a summary of the test results. The minimum yield load for a plain pile, 1a or 1b, was 87 tons, and the maximum, 165 tons, in the accelerated tests. For the core-stoppered piles the yield loads varied from 35 tons to 104 tons, with one exception, A2d, which carried 147 tons. The average yield load for all core-stoppered piles was about 77 tons, and for the plain piles, 121 tons. For a factor of safety of 1½, corresponding to an overload of 50 per cent, the design load would vary from 23 to 69 tons for the care-stoppered piles, and from 58 to 110 tons, with an average of 81 tons, for the plain piles.

The average direct unit stress for an 81-ton load is only 8,600 pounds per square inch, leaving ample margin for any bending stresses

TABLE 1. — BAY BRIDGE PILE TESTS. PILE DRIVING DATA

PILE DRIVING DATA

Pile No.	Bent No.	Location in Bent	Length Ft.	Original Driving					Re-Driving			
				Date	Total Penetration Ft.	Penetration Under Own Wt. Ft.	Total No. of Blows	Av. Penetration per Blow Last Foot Driving In.	Date	Additional Penetration Ft.	Total No. of Blows	Av. Penetration per Blow Last Inch Driving In
A1a	L4S	2 W	99.5	7-5-35	41.7	18.9	1007	.081	7-31-35	3.00	326	.131
A1b		2 E	99.0	7-5-35	41.4	17.0	1598	.051	7-31-35	3.00	481	.083
A2a		4 W	95.2	7-5-35	32.1	14.6	511	.856	8-2-35	3.00	105	.083
A2b		7 W	89.3	7-5-35	32.7	15.2	676	.145	8-1-35	3.00	194	.167
A2c		9 W	99.1	7-5-35	34.3	15.8	147	1.200	7-30-35	2.75	141	.204
A2d		4 E	98.8	7-8-35	32.7	15.3	633	.231	8-2-35	3.00	427	.050
A2e		7 E	98.7	7-8-35	30.8	15.3	664	.145	8-1-35	3.00	213	.167
A2f		9 E	94.4	7-8-35	33.3	15.5	760	.324	7-31-35	3.00	142	.279
B1a	L4N	2 W	100.1	7-3-35	41.3	17.8	1449	.061	8-4-35	.17	46	.050
B1b		2 E	99.6	7-6-35	40.3	16.8	545	.146	8-4-35	.17	47	.056
B2a		4 W	97.2	7-3-35	33.7	16.8	319	.333	8-3-35	.17	14	.143
B2b		7 W	99.8	7-5-35	34.3	17.8	279	.387	8-3-35	.17	18	.125
B2c		9 W	93.6	7-5-35	33.8	17.3	170	.856	8-2-35	.17	25	.071
B2d		4 E	94.0	7-5-35	33.2	16.2	366	.333	8-3-35	.17	15	.143
B2e		7 E	99.0	7-5-35	33.8	16.3	327	.333	8-4-35	.17	14	.143
B2f		9 E	101.5	7-5-35	33.3	16.3	399	.324	8-3-35	.17	22	.111
C1a	L2S	2 W	99.2	7-10-35	43.2	19.5	491	.364				
C1b		2 E	98.8	7-10-35	40.5	17.6	663	.141				
C2a		4 W	111.2	7-10-35	34.7	19.4	386	.445				
C2b		7 W	106.5	7-10-35	35.8	20.5	227	.500				
C2c		9 W	109.6	7-10-35	34.8	20.5	370	.286				
C2d		4 E	111.2	7-10-35	30.9	16.9	464	.387				
C2e		7 E	111.6	7-10-35	31.3	16.5	384	.245				
C2f		9 E	109.2	7-11-35	31.9	17.3	387	.375				
D1a	L2N	4 W	99.7	7-8-35	40.9	17.0	623	.316				
D1b		4 E	99.1	7-8-35	41.2	17.9	564	.218				

All Piles 12 WF 65.

For number and location of core-stoppers on Piles in Bents L4S, L4N and L2S, see Fig. 7.

All Piles driven with Mc Kiernan-Terry Double Acting Steam Hammer No. 11-B-2. Average steam pressure at boiler 118#/□". Min. pressure 100#/□". Assumed average energy per blow, 19,150 ft. lbs.

TABLE 2. — BAY BRIDGE PILE TESTS. DRIVING DATA AND TESTING RESULTS

DRIVING DATA AND TESTING RESULTS

Pile No.	Bent No.	Location in Bent	Length	Penetration	Penetration Last Blow	Accelerated Load Test			Pulling Test			Sustained Loading Test				Point Resistance		
						Yield Load	Settlement of Y. L.	Ultimate Load	Starting Load	Maximum Load	Frictional Resistance	Sustained Load	Period of Loading	Settlement under S. L.	Ultimate Load	Ultimate - Max. Pull Load	% Yield Load	% Ult. Load
			Ft.	Ft.	In. ‡	Tons	Ins.	Tons	Tons	Tons	#/□"	Tons	Hrs.	In.	Tons	Tons	%	%
A1a	L4S	2W	99.5	44.7	.131	86.6	.390	99.6	38.0	118.8	6.30	*				—	—	—
A1b		2E	99.0	44.4	.083	147.2	.663	153.7	28.5	133.1	7.08					20.6	14	13
A2a		4W	95.2	35.1	.083	69.3	.446	129.9	28.5	39.2	2.65					90.7	130	70
A2b		7W	89.3	35.7	.167	60.6	.280	77.9	28.5	57.0	3.78					20.9	35	27
A2c		9W	99.1	37.0	.204	60.6	.398	86.6	—	—	—					—	—	—
A2d		4E	98.8	35.7	.050	147.2	.788	190.5	28.5	47.5	3.16					143.0	97	75
A2e		7E	98.7	33.8	.167	69.3	.458	77.9	26.0	54.6	3.84					23.3	34	30.
A2f.		9E	94.4	36.3	.279	69.3	.429	90.9	28.5	47.5	3.10					43.4	63	48
B1a	L4N	2W	100.1	41.4	.050	138.6	.671	151.5	38.0	118.8	6.79					32.7	24	22
B1b		2E	99.6	40.4	.056	164.5	.856	179.7	38.0	113.4	6.65					66.3	40	37
B2a		4W	97.2	33.8	.143	103.9	.426	119.1	28.5	52.2	3.66					66.9	64	56
B2b		7W	99.8	34.4	.125	95.2	.370	110.4	30.9	61.7	4.25					48.7	51	44
B2c		9W	93.6	34.0	.071	103.9	.493	116.9	14.2	67.7	4.70					49.2	47	42
B2d		4E	94.0	33.3	.143	95.2	.514	116.9	—	47.5	3.38					69.4	73	59
B2e		7E	99.0	33.9	.143	95.2	.475	116.9	28.5	62.9	4.39					54.0	57	46
B2f		9E	101.5	33.5	.111	86.6	.468	103.9	—	51.0	3.60					52.9	61	51
C1a	L2S	2W	99.2	43.2	.364	86.6	.288	95.2	38.0	81.4	4.46	69.3	14.3	.015	101.7	13.8	16	15
C1b		2E	98.8	40.5	.141	103.9	.536	112.5	38.0	81.9	4.78	77.9	12.9	.041	103.9	30.6	29	27
C2a		4W	111.2	34.7	.445	51.9	.225	60.6	19.0	47.5	3.26					13.1	25	22
C2b		7W	106.5	35.8	.500	60.6	.307	68.2	20.2	47.5	3.14	47.6	15.0	.035	64.9	20.7	34	30
C2c		9W	109.6	34.8	.286	69.3	.425	77.9	38.0	52.2	3.56					25.7	37	33
C2d		4E	111.2	30.9	.387	43.3	.157	59.5	19.0	35.6	2.73	38.9	14.0	.179	-	23.9	55	40
C2e		7E	111.6	31.3	.245	69.3	.345	77.9	28.5	47.5	3.58					30.4	44	39
C2f		9E	109.2	31.9	.375	34.6	.175	56.3	30.9	32.1	2.38					24.2	70	43
D1a	L2N	4W	99.7	40.9	.219	—	—	—	19.0	89.1	5.16	43.3	13.8	.018	62.8	—		
D1b		4E	99.1	41.2	.152	—	—	—	28.5	102.2	5.89	38.9	13.3	.081	56.3	—		

* Sustained Load = roughly 80% of Yield Load and 70% of Ultimate Load as determined by Accelerated Loading Test.

‡ Piles driven with Mc Kiernan-Terry No. 11-B-2 hammer - Assumed energy, 19150 ft. lbs. per blow.

due to accidental eccentric loading, or it would provide for a reduction in stress corresponding to a column having a slenderness ratio of 140. For the maximum safe design load of 110 tons the unit stress would be 11,500 pounds per square inch, and for the maximum yield load of 165 tons, 17,300 pounds per square inch, or greater than would ordinarily be allowed on even a short column. It is obviously quite possible, therefore, to obtain a yield load on a plain steel H-pile greater than its capacity as a column, and a safe design load equal to that capacity, without excessive penetration in ordinary sandy-clay soils.

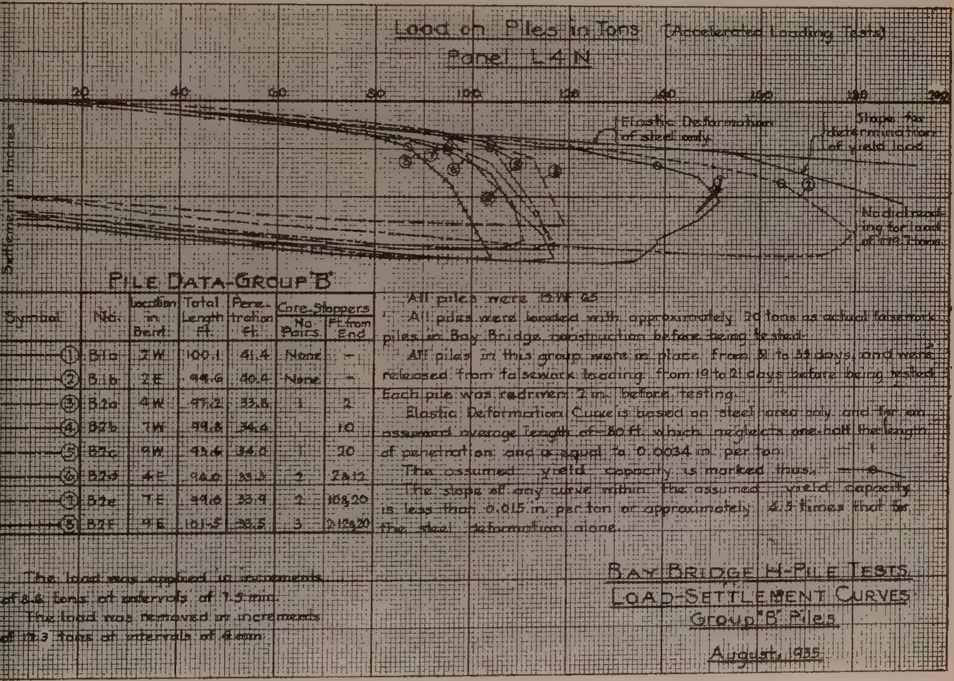
A graphical comparison of the yield loads is shown in Fig. 4 for the three groups A, B and C, also the detail and location of the core-stoppers and a boring profile. With one exception the superiority of the plain piles over the core-stoppered sections in each group is plainly shown. These diagrams also show that the location of the core-stoppers seems to have little effect, but from group C it would seem that they are more effective the higher up they are placed.

A comparison of the yield capacity of the three groups of piles is very instructive, and shows that for both the plain and core-stoppered piles the average load is greatest for group B and least for group C. This is rather surprising and not at all what was expected. It was supposed that group C piles, which were not redriven, would test most favorably, and that group A piles, redriven 3 feet, would make the poorest showing. Group B piles, redriven only 2 inches, gave the best test results by a wide margin. The reason for these phenomena is not known, and it would be interesting to see if further tests show similar results.

As to the use of core-stoppers, it is obvious from these tests that any attachment which breaks the continuity of the surface of the imbedded length of a pile is detrimental to the load-carrying capacity of that pile. Such a device apparently destroys the frictional value of the soil against the pile, and that friction is not restored in as much as four weeks' time. Even those core-stoppers located 20 feet from the tip, or just at the bottom of the 13-foot bed of surface mud, where they should have done the most good, were of practically no help. A plain pile driven 7 to 10 feet farther into a bed of nearly pure clay was of much greater capacity.

The action of these core-stoppers, and the behavior of the piles generally, is indicated by a study of the load-settlement curves. In group A piles were redriven 3 feet before testing. Pile No. 6, A2d, is the only core-stoppered pile which equaled or exceeded the capacity of a plain pile. It is believed that this pile may have struck a boulder or some sort of hard spot, and it certainly is not comparable to any of the other piles. Pile No. 1, A1a, one of the plain piles, is likewise not representative and the gauges are believed to have slipped.

Fig. 5 shows the data for group B piles which were redriven 2 inches, and it is a remarkably consistent set of curves. The two plain piles, 1 and 2, show not only a greater capacity but a more gradual settlement under their maximum loads. The core-stoppered piles failed not only at lesser loads, but more abruptly than the plain piles, and generally settled a greater amount under the same loads even within the yield capacities. The close bunching of these curves is quite remarkable, and is evidence of the care and accuracy with which the tests were conducted.



PILE DATA-GROUP "B"

Symbol	No.	Location in Bent	Total Length Ft.	Penetration Ft.	Core Stoppers	
					No. Pairs	Ft. from End
①	B1a	2W	100.1	41.4	None	–
②	B1b	2E	95.6	40.4	None	–
③	B2a	4W	97.2	35.8	1	2
④	B2b	7W	99.8	34.6	1	10
⑤	B2c	9W	93.6	34.0	1	20
⑥	B2d	4E	94.0	35.3	2	2&12
⑦	B2e	7E	99.6	33.9	2	10&20
⑧	B2f	9E	101.5	38.5	3	2,12&20

All piles were 12W 65

All piles were loaded with approximately 30 tons as actual falsework piles in Bay Bridge construction before being tested.

All piles in this group were in place from 31 to 33 days, and were released from falsework loading from 19 to 21 days before being tested.

Each pile was redriven 2 in. before testing.

Elastic Deformation Curve is based on steel area only and for an assumed average length of 80 ft. which neglects one-half the length of penetration and is equal to 0.0034 in. per ton.

The assumed yield capacity is marked thus. —◆—

The slope of any curve within the assumed yield capacity is less than 0.015 in. per ton or approximately 4.5 times that for the steel deformation alone.

The load was applied in increments of 8.6 tons at intervals of 1.5 min.

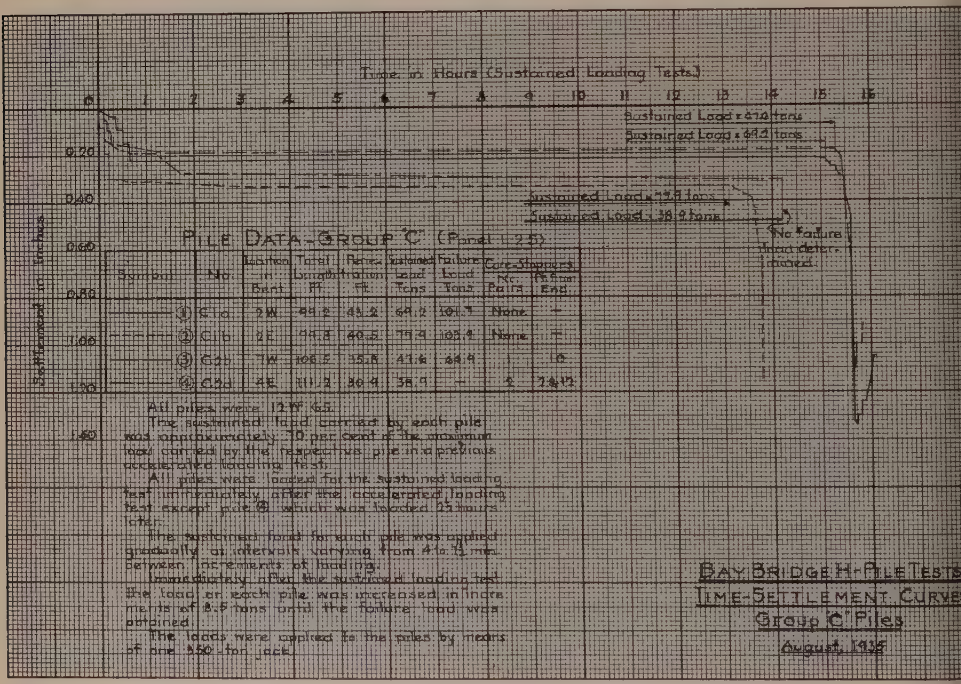
The load was removed in increments of 13.3 tons at intervals of 4 min.

BAY BRIDGE H-PILE TESTS
LOAD-SETTLEMENT CURVES
Group "B" Piles
August, 1935

FIG. 5. — BAY BRIDGE H-PILE TESTS. LOAD-SETTLEMENT CURVES FOR GROUP "B" PILES

The recovery curves are about a straight line and nearly parallel to the elastic deformation line for steel only. The permanent settlement of one to one and one quarter inches is, for the plain piles, well within the one hundredth of an inch settlement per ton of load sometimes used as a criterion for a test load. For the core-stoppered piles the settlement is a little excessive.

The data for group C piles, those which were not redriven before testing, show much more erratic results, particularly for the core-stoppered piles, than either of the other groups, and proportionately greater permanent settlements.



PILE DATA-GROUP "C" (Panel 1-25)

Symbol	No.	Location in Bent	Total Length Ft.	Penetration Ft.	Sustained Load Tons	Failure Load Tons	Core-Stoppers No. Pairs	Core-Stoppers Ft. from End
①	C1a	2W	99.2	43.2	69.2	101.7	None	–
②	C1b	2E	99.8	40.5	77.9	103.4	None	–
③	C2b	7W	106.5	35.8	41.6	64.9	1	10
④	C3d	4E	111.2	30.4	38.9	–	2	2 & 12

All piles were 12 W 65.

The sustained load carried by each pile was approximately 70 per cent of the maximum load carried by the respective pile in a previous accelerated loading test.

All piles were loaded for the sustained loading test immediately after the accelerated loading test except pile ④ which was loaded 23 hours later.

The sustained load for each pile was applied gradually at intervals varying from 4 to 11 min. between increments of loading.

Immediately after the sustained loading test the load on each pile was increased in increments of 8.5 tons until the failure load was obtained.

The loads were applied to the piles by means of one 150-ton jack.

FIG. 6. — BAY BRIDGE H-PILE TESTS. TIME-SETTLEMENT CURVES FOR GROUP "C" PILES

Fig. 7 shows the comparison of the accelerated and sustained loading tests for four piles, two plain and two with core-stoppers, which were tested both ways. The accelerated load test was made first, followed by a sustained load of about 70 per cent of the maximum or 80 per cent of the yield load first determined. In all four cases the piles held this sustained load with about the same settlement as in the accelerated test. The load was then increased to failure, and the load-settlement curves agreed quite closely with those of the accelerated tests, with one exception. Pile C2d, a core-stoppered pile, failed very rapidly with the addition of further load after the sustained load test, indicating again the unreliability of the use of core-stoppers. The other core-stoppered pile and one of the plain piles, C1a and C2b, showed exactly the same yield load as in the accelerated test. The other plain pile, C1b, showed about 85 per cent of the yield load in the sustained test that it did in the accelerated test, and about 92 per cent of the ultimate load.

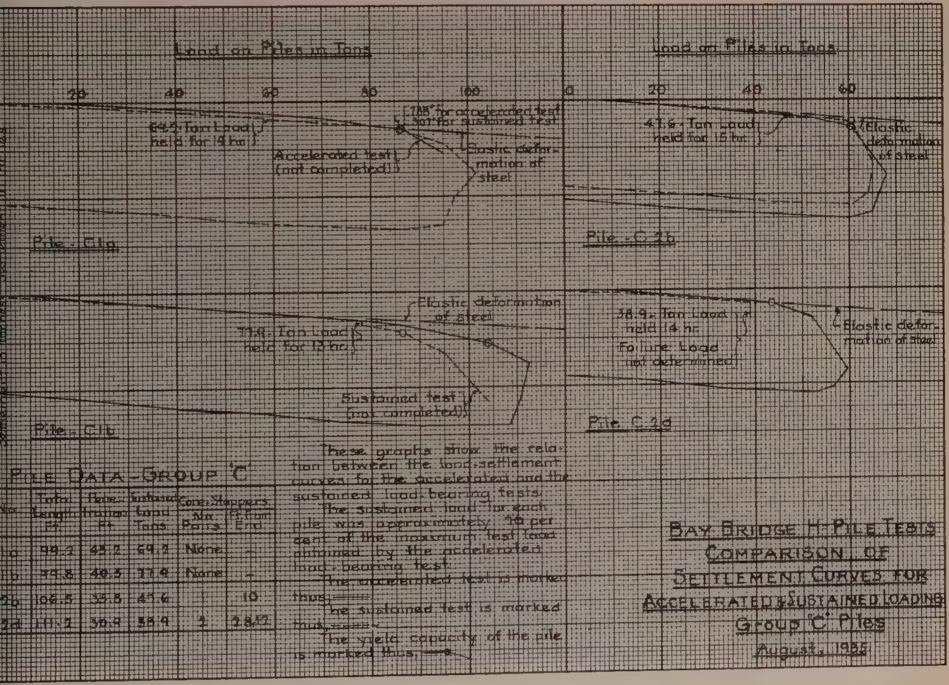


Fig. 7. — Bay Bridge H-Pile Tests. Comparison of Settlement Curves for Accelerated and Sustained Loading for Group "C" Piles

Another interesting sustained loading test was conducted on two plain piles referred to as D1a and D1b. These were not included in the preliminary load-bearing tests after their use as falsework piles, but were first pulled about 8 inches each to break them loose and destroy the point bearing. The load required to start pulling them was only 19 and 28.5 tons, respectively, although their maximum resistance was 89 and 102 tons. Their pull-yield curves (not shown here) indicate a definite break at about 75 and 65 tons. The testing engineer arbitrarily chose loads of 43 and 39 tons with which to test them as pure friction piles, since it was believed they could have little other resistance. These loads correspond to 3.5 and 3.2 pounds per square inch friction on the actual perimeter of the pile. These loads were held for nearly fourteen hours. One of the piles showed an initial settlement of only .015 of an inch, and no further settlement occurred during the fourteen-hour period. The other pile settled about .06 of an inch in the first hour and about .02 of an inch additional in the next thirteen hours. After the fourteen-hour period the load was gradually increased to failure, the ultimate loads carried being 63 and 56 tons, respectively, for which the net settlement was .42 of an inch and .55 of an inch, and which was still within the limit allowed for a "safe" test load by the New York Building Code. These tests are regarded as particularly valuable, since the resistance must have been due entirely to friction and cohesion in the soil. From the

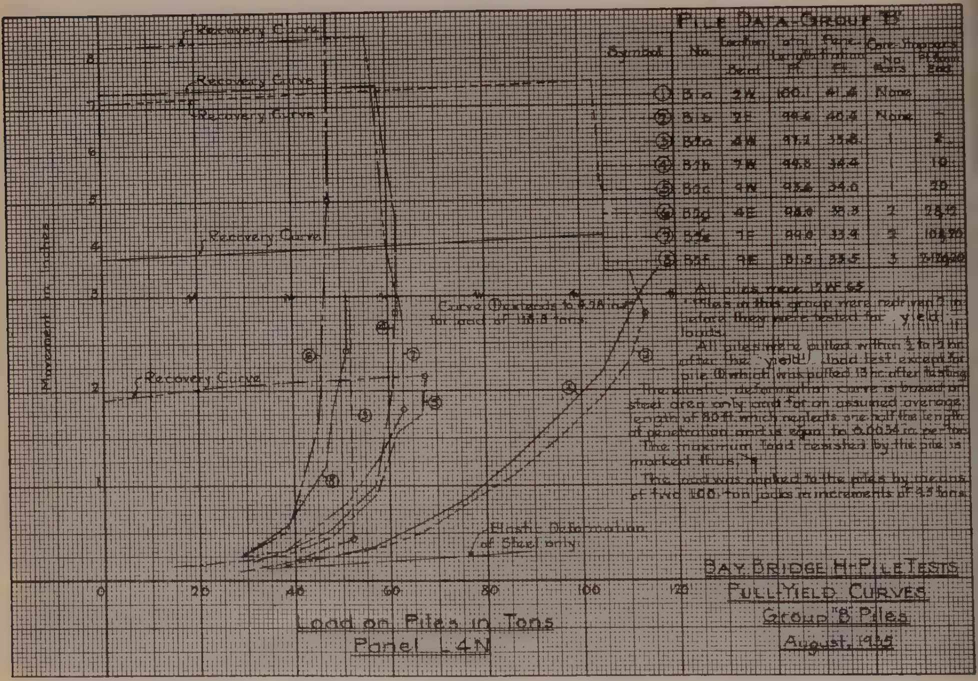


FIG. 8. — BAY BRIDGE H-PILE TESTS. PULL-YIELD CURVES FOR GROUP "B" PILES

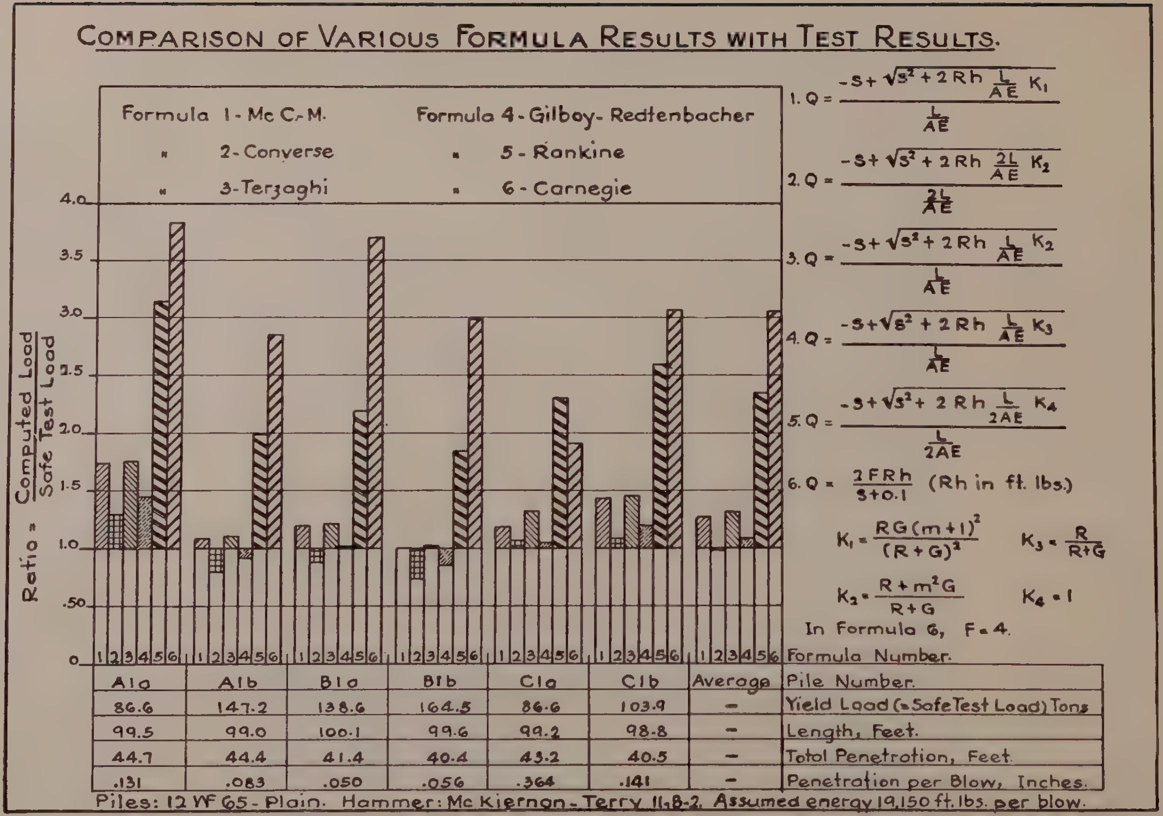
nature of the soil there is little doubt but that an actual hole existed immediately under the end of the pile. So far as is known this is the only test of its kind which has been made.

The results of the pulling tests for the B-piles are shown by the pull-yield curves in Fig. 8. The outstanding characteristic is the fact that the plain piles required a very much greater load to pull them, from two to two and one half times, than did the core-stoppered piles. The maximum pulling loads varied from 81 to 133 tons for the plain piles, and from 32 to 68 tons for the core-stoppered piles. The corresponding frictional values are 4.5 to 7.1 pounds per square inch for the former, and 2.4 to 4.7 pounds per square inch for the latter. The lowest value for a core-stoppered pile amounts to 1 ton per foot, and for a plain pile, almost 2 tons per foot. The highest values for each were about 2 tons and 3 tons, respectively. These values justify the statements made previously as to frictional values to be expected. The real problem of determining safe loads on piles in such soils is really one of possible settlement and flow of the soil itself rather than of the frictional value of the pile in the soil.

One of the most important objectives of this test program was to make comparisons between the test loads and those obtained by various pile-driving formulas. While it is generally freely admitted that even rational pile-driving formulas are not dependable enough to rely upon for establishing safe loads for steel piles, there is no other way of determining design loads except by test. Where the latter is not possible, and it is not for every pile driven on a job, it is then customary to turn right around and use a pile-driving formula, and usually our old friend, the Engineering-News formula itself. Why this should be, when there are better formulas available, is a queer thing. It may have been good enough in the days of timber piles and 15-ton loads, but it has no place with steel piles and greater loads, as its originator would probably agree.

First, what is a rational type of pile-driving formula? No formula can be considered rational which does not take account of the fact that steel is not a perfectly elastic material. Its coefficient of restitution or of the elasticity of impact, designated as m, is only .55, and not unity, and tests have shown that any formula which omits this coefficient gives values which are too large.

A comparison of results obtained by the use of several formulas of the rational type is shown in Fig. 9, applied to the plain test piles only. The first three take account of the coefficient m in the factor k. The fourth omits m as such but retains a k value based on the relative weight of ram and pile. The chart shows the bearing values by the formula as a



$$1.\ Q = \frac{-S + \sqrt{S^2 + 2Rh\frac{L}{AE}K_1}}{\frac{L}{AE}}$$

$$2.\ Q = \frac{-S + \sqrt{S^2 + 2Rh\frac{2L}{AE}K_2}}{\frac{2L}{AE}}$$

$$3.\ Q = \frac{-S + \sqrt{S^2 + 2Rh\frac{L}{AE}K_2}}{\frac{L}{AE}}$$

$$4.\ Q = \frac{-S + \sqrt{S^2 + 2Rh\frac{L}{AE}K_3}}{\frac{L}{AE}}$$

$$5.\ Q = \frac{-S + \sqrt{S^2 + 2Rh\frac{L}{2AE}K_4}}{\frac{L}{2AE}}$$

$$6.\ Q = \frac{2FRh}{S+0.1} \quad \text{(Rh in ft. lbs.)}$$

$$K_1 = \frac{RG(m+1)^2}{(R+G)^2} \qquad K_3 = \frac{R}{R+G}$$

$$K_2 = \frac{R+m^2G}{R+G} \qquad K_4 = 1$$

In Formula 6, $F = 4$.

A1a	A1b	B1a	B1b	C1a	C1b	Average	Pile Number.
86.6	147.2	138.6	164.5	86.6	103.9	–	Yield Load (=Safe Test Load) Tons
99.5	99.0	100.1	99.6	99.2	98.8	–	Length, Feet.
44.7	44.4	41.4	40.4	43.2	40.5	–	Total Penetration, Feet.
.131	.083	.050	.056	.364	.141	–	Penetration per Blow, Inches.

Piles: 12 WF 65 - Plain. Hammer: Mc Kiernan - Terry 11-B-2. Assumed energy 19,150 ft. lbs. per blow.

FIG. 9. — BAY BRIDGE H-PILE TESTS. COMPARISON OF VARIOUS FORMULA RESULTS WITH TEST RESULTS

ratio of the yield load determined by test, and for the first four formulas the results are very gratifying. Formulas 2 and 4 gave results within 45 per cent of the test result in all cases, and formulas 1 and 3 were nearly as satisfactory. Formula 5, which is of the rational type but uses $k=1$, shows how violently the values are changed, being two to three times too large. Formula 6 is of the Engineering–News type modified by a factor F in the numerator intended to take account of the type of soil into which the pile is driven and which must be judged largely by the penetration per blow. The results shown by the use of this formula are two and one half to nearly four times too large, but may be challenged by the contention that the wrong value of F was used. If we know the answer in advance, however, any one could guess the correct value, and there would be no need for a formula. The difficulties and doubts connected with its use are quite obvious.

Reference to the chart will show that if any of the first four formulas had been used to evaluate the bearing value from the driving data a factor of safety of 2 would have been ample to keep the design load within the yield capacity. The best showing was made by Professor Converse's variation of the formula, the results varying from 63 per cent to 130 per cent of the yield load. These are only a few of the tests to which formulas of this type have been applied, and the results have been uniformly satisfactory. In the case of clay soils, however, in which the bearing value of the pile tends to increase with a period of rest after driving, the test blows to be used in the formula must be made after the piles have had a chance to "set up." Some objection has been made that this is not a suitable or scientific method, but the cases in which it has been used so far have given satisfactory results.

The following brief summary may be made of the results of the foregoing tests of steel H-piles driven in clay and clay-sand soils:

1. The yield load of the plain piles varied from 86 to 147 tons, an average of 121 tons. The average penetration was about 42½ feet.

2. The use of core-stoppers on an otherwise plain pile was shown to be very detrimental to the capacity and general stability of that pile. The average yield load carried by the plain piles was 121 tons and only 77 tons for the others, or 57 per cent greater for the plain piles.

3. The results obtained from the fourteen-hour sustained loading tests compared favorably with the accelerated test results and justified the accelerated method of testing.

4. The maximum frictional resistance of the plain piles, as determined by pulling tests, varied from 4.5 to 7.1 pounds per square inch, averaging 6.0 pounds per square inch. For the core-stoppered piles the

values were 2.7 to 4.7, averaging 3.5 pounds per square inch. The plain piles averaged over 70 per cent better frictional value.

5. These tests demonstrated that bearing values can be obtained from pile-driving data by means of a rational type of formula that will give results within 30 per cent of the yield load (not the maximum load) as determined by test. Such results used with a factor of safety of 2, or at the most, 2½, can be expected to give an actual factor of safety of not less than 1½ as would be used with an actual test load.

THE FACTOR OF SAFETY IN HIGHWAY DESIGN

By Charles M. Noble *

(Presented at a meeting of the Boston Society of Civil Engineers held on January 27, 1937)

It is with some reluctance that I discuss the safety factors entering into highway design because of the lack of knowledge of many of the fundamentals involved, and yet it is this very ignorance which must be examined if we are to throw light on a rather obscure problem. The shadows of doubt must be dispelled, and there must be an appreciation of the problem before highway engineers will be on the road to safe design.

By safe design, as used herein, is not meant the structural integrity of the pavement or other materials of construction, but that the condition of the highway will be safe for the vehicles which operate upon it under all weather and traffic conditions.

The designer of a bridge, for example, would not dream of proportioning the members based on the ultimate strength of the materials used, nor would he dream of designing the structure for loads less than those he knew it would be called upon to support. And yet, very frequently, the highway designer is doing just that. Again it must be emphasized that it is not the structural design of the materials of construction that is discussed, but it is the more intangible type of design which merits attention, the defects of which are responsible for a large percentage of accidents. Traffic density and speed are the loads. Alignment, grades, super-elevation, sight distance, shoulders, signs, lighting and a multitude of details which add to vehicle safety are the materials. The fault has been of assuming loads that were too small and then stressing the materials up to and beyond their ultimate strength. In no other field of engineering is this done. It is axiomatic that success in engineering design is dependent on the correct assumption of the loads to be carried, the proper selection of the materials of construction, and the determination of the correct stress to apply to these materials. It is the solemn duty of engineers to so design the highway that it will be safe from an operating standpoint. If 36,000 people were killed each year by

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the failure of buildings, dams and bridges, the engineers who designed these structures would find themselves in an untenable position. But because of the intangible nature of the problem, the highway engineer has gone practically unchallenged. Yet there are indications that the public is awakening to the situation. It is therefore essential for the engineer charged with highway design to be accident conscious. Many defects are obvious, but there are many faults which are unknown, because there has not been an exhaustive scientific analysis of the basic causes of accidents. Some subtle fault in the highway of which the driver is not aware may be the originating cause of an accident. Faulty lighting is a simple illustration. It will be necessary to discover all the basic and contributing causes of accidents before the technic of safe highway design is perfected, and it will be necessary for the designer to cultivate the proper attitude of mind. The question arises as to whether the safety problem is attacked in the searching, scientific way that is followed in other engineering problems. Modern highway design started about twenty years ago. Since that time great progress has been made in the design of the materials of construction, but what accomplishments have really been achieved in the realm of safety? Our profession is reputed to be conservative, but, as applied to highways, this conservatism has been applied to the expenditure of money rather than to the provision of adequate factors of safety. Conventional standards have been adopted relating to curvature, super-elevation, maximum gradients, etc., but to what extent may safety be built into the highway?

Many no doubt think that it is unnecessary to provide higher safety standards; that the solution lies in the control of speed and of the operator. It is probably safe to say that nearly all of us deprecate the high speeds obtaining today and feel that speeds approximating 50 miles per hour are high enough; but the fact remains that the vast body of automobile owners are not of that opinion, and it is they who will dictate the speeds which *will* obtain. Within the next ten years the problem will relate to sustained speeds of from 80 to 100 miles per hour. Speed control has been seriously proposed several times during the past twenty years, but the movements have died because public sentiment has not supported the idea. Thus engineers are faced with speed and operating conditions over which they have no control, and the obvious procedure from an engineering standpoint is courageously to face this problem and make a sincere and intelligent effort to design safety into the highway.

Again, many are wondering if we can afford to spend the money to make the highways safe. It is my belief that we cannot afford to economize on this element of design. Suppose for a moment that this matter is considered from a cold-blooded economic standpoint. The first point to consider is the matter of obsolescence. The designers of other engineering works would not feel that they had much foresight if they were compelled to rebuild their structures every ten or fifteen years. Yet it is common knowledge that many of the highways located fifteen years ago are in need of rebuilding because of inadequate safety and operating standards. This is a tremendous economic waste, not only because of the unnecessary added expenditure of funds, but from the standpoint of landowners affected, who must readjust themselves each time the highway is relocated. Many farms have been unduly penalized by being cut up into unusable or uneconomical parcels. It has been said that the right of way is the only permanent feature of a highway. If properly located for alignment and width, it should remain a fixture for generations. But unfortunately this has not been general. More often the road has been reconstructed because of obsolescence of the right of way, while the pavement and other structures still were capable of rendering efficient service. Doubtless an economist could make a rather sorry picture of this form of economics. That wasted effort and money is coming directly out of the cash drawer of highway expenditures. The second point is more intangible, but, nevertheless, just as real, namely, the economic loss due to deaths, hospital and doctors' bills, loss of time from injuries, and damage to cars. No one has made any accurate appraisal of such loss. The Travelers Insurance Company of Hartford estimates that it amounts to between two billion and two and one half billion dollars each year. Other authorities * have stated that statistics indicate that 30 per cent of all accidents may be traced to some fault in road location and design. This indicated that faulty highway design is costing the American public from six hundred million to seven hundred and fifty million dollars each year. This is 60 per cent of what has been estimated we should spend annually for new construction. If, then, it is economically justified to spend 60 per cent more for each mile of highway than would normally be spent, it is evident that the designer has considerable latitude financially in providing fundamental safety in his designs. Strictly, such a large added expenditure may not be justified, but if there is added to the balance sheet the factor of obsolescence, and the humanitarian elements are considered, it is quite evident that we *can* afford to become accident conscious.

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* Engineering News-Record, October 29, 1936, p. 612.

In defence of highway locators and designers of fifteen years ago, let it be remembered that they were literally pioneering in a new form of transportation. A speed of 35 miles per hour was considered a very fair driving speed. No one really dreamed of a stock car of moderate price that could travel with ease hour after hour at 75 or 80 miles per hour. The problem of that day was to get the main road system out of the mud. The demand was for a large mileage of cheap roads so that the motorist might have some use for the car after it was bought. A road system was needed and needed quickly in order that the motor industry might develop. At that time the revenues derived from the motorist were comparatively modest, and the taxpayers expected a road dollar to go a long way geographically. The user did not worry about sharp curves, short choppy grades, narrow width or other defects. He was content to be out of the mud. Under such circumstances it is only natural that most of those roads have been or shortly will be reconstructed to higher standards; yet, in spite of this, it is fitting to salute the men who accomplished the mammoth task of pulling America out of the mud.

Today the picture is very different from that of 1920. The pioneering period has definitely closed, and the highway and motor industries are entering upon an era of early maturity. The design of the motor car has been improved to a state of excellence undreamed of in that day. This rapid advance in design is one of our modern miracles, and high tribute is due the genius of automotive engineers and manufacturers. At the same time, the highway industry is no longer a poor cousin. It is one of the great leaders of industry. The sums spent for highway improvement are being largely contributed by the user instead of being taken from the property owner's pocket. That these sums are considerable, and that highway development has passed from the pioneering stage, is strikingly illustrated by figures compiled by the American Automobile Manufacturers Association and the Petroleum Institute.

In 1920 the vehicle registration was 9,232,000; in 1935, 26,221,000, or three times greater.

In 1920 the gasoline consumption was 4,200,000,000 gallons; in 1935 it was 17,160,000,000 gallons, or four times greater.

In 1920 the revenue derived from automobile license fees, gasoline and oil taxes was $103,910,000; in 1935 it was $958,753,000, or nine times greater.

In addition, the manufacturers' and personal property tax was $350,000,000.

The appalling 1935 death record of 36,000 lives and 895,000 injured further indicates that highway accidents also have reached a gruesome maturity.

Thus it can be seen that engineers may well pause and take sober thought as to the lines upon which the highway should develop.

The thought which now presents itself is, in what way should design be improved in order to provide real basic safety. As stated earlier many factors are unknown which fundamentally are the cause of accidents. Some of these are quite subtle; research, observation and analysis will be required to throw the light of definite knowledge upon the dark places of our ignorance.

But there is one factor which is a key to many of these problems, namely, *time*. Human reaction has a definite time limit. It takes just as long today to react to overcome an error in driving judgment or to adjust one's self to a road hazard as it did in 1920. But in 1920 we would probably have been driving 30 miles per hour, while today one may be driving 70 miles per hour. Then a distance of 44 feet per second was covered, but today 102 feet per second; yet unfortunately our reaction time to danger has remained constant. This, then, imposes on the designer the task of lengthening out the perspective ahead of the driver, and above all, to impose no sudden change in driving conditions. In order to keep this element of time, this fourth dimension, if you please, ever present in our thinking, particular attention should be given to velocities in feet per second for different rates of speed, and constant use made of these data as a measuring stick in solving safety design problems.

Another factor which has become quite evident is the extreme danger of high speed traffic traveling in opposite directions with no space between. This presents a serious problem where present traffic does not require more than two lanes. The only apparent method of providing improved safety for such a condition is to use extra width lanes, wide, smooth shoulders, and sight distances greater than 2,000 feet in order that passing may not be unduly hazardous. For the condition where four lanes are required the solution is simple, — divided roadways. This type of construction is receiving increased attention, but there is some difference of opinion as to the details of design.

Consider some of the details of design which must receive attention because of the *time* factor. It would appear desirable to design on the basis of a constant speed. That is to say, after a careful consideration of all the factors of use, type of road, future development and topography, a design speed is selected. When this is done one of the unknown elements has been eliminated, and it is then possible to proceed in a logical way to work out details so that all the elements are arranged uniformly, consistently and logically to conform to that speed. The motorist should not be subjected to the danger of suddenly encountering a set of conditions requiring a much lower speed than required for other portions of the road. Such sudden changes in speed requirements are a prolific source of accident. If it becomes necessary to change the speed requirements, such changes should be done gradually in order to give the motorist *time* to sense the change in driving conditions. In considering the setting up of design speeds it is manifestly absurd to set up the same speed for a farm road as for a main line trunk highway, and it is this determination which requires the utmost in judgment, experience and foresight. Upon such determination will depend the location and width of the right of way. With a known speed it is possible to think in terms of time. The car travels so many feet per second and requires so many feet to be brought to a stop. A logical set of standards can be set up for sight distance along the road and at intersections, curvature, width of pavement and shoulders, spacing and arrangement of direction and warning signs, length of profile tangents and vertical curves, adequate warning of unusual conditions, all to conform to the design speed.

Yet, setting up such standards is not quite so simple as would appear at first thought. Unfortunately, there are differences of opinion relative to a number of fundamental questions. A casual glance at some of these details may be of interest.

Most designers admit that adequate sight distance is important, but just what constitutes adequate sight distance is not so clear. For a two-lane road passing requirements will control. It is possible to compute the distance consumed in passing if the speeds of the passing car, the car being passed, and an approaching car are known; and it is also possible to assume certain speed combinations which indicate that a sight distance of thousands of feet is required. Thus it is evident that more knowledge is essential before a proper standard can be set up. In the case of divided roadways some sort of measure of sight distance can be arrived at by considering the distance required to bring a vehicle to a stop.

Fig. 1 shows the distance required to bring a vehicle to a stop after the brakes are applied, based on a deceleration factor of 17.4 feet per second per second. A greater distance is required on a descending grade, less on an ascending grade. To the distance taken from the diagram must be added the distance traveled during the lag period. The lag period is made up of two reaction times, — first, the interval required for the driver to realize the necessity for stopping; second, the time required to apply the brakes. The lag period may vary from three quarters to one and one half seconds. For convenience the distances in feet per second are shown in this part of the diagram. It is not recommended that this particular diagram be used to determine sight distance, because it is based on the braking power of the car, such braking power often being greater than the actual coefficient of friction between tires and road surface. A deceleration factor of 17.4 requires a friction coefficient greater than 0.5. Wet surfaces may have less than this, and icy surfaces usually have a coefficient only slightly above 0.1. This figure is shown for illustrative purposes only.

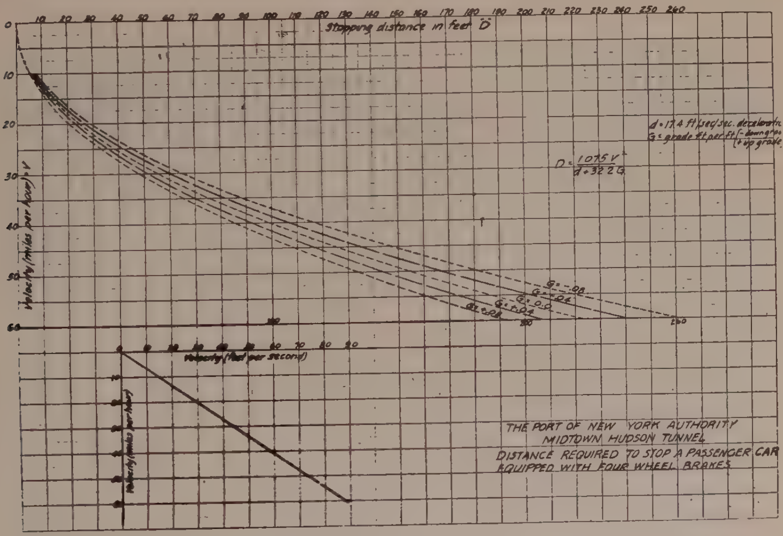


FIG. 1. — DISTANCE REQUIRED TO STOP A PASSENGER CAR EQUIPPED WITH FOUR-WHEEL BRAKES

At intersections the sight distance required will depend on the speed of approaching vehicles and the time required by drivers to adjust themselves to conditions. Here, too, there is no definite knowledge, but it would seem reasonable to use nearly the same amount required as for the previous condition.

The minimum radii of curves appear to be related to the maximum super-elevation permitted and the speed. The radius is obtained by the formula $E+f=.067\frac{v^2}{R}$, from which $R=\frac{.067\ v^2}{E+f}$. Thus it is possible to determine the minimum radius based on the design speed and the maximum banking permissible. This is a somewhat more logical approach than the arbitrary methods of the past. Such determination is facilitated by the use of a diagram.

Fig. 2 shows the relation of speed to super-elevation to radius for the portion of the formula $E = \frac{.067\, v^2}{R}$. This makes necessary consideration of the factor f, which is the permissible (side) friction coefficient between tires and road surface. As yet there is no definite agreement as to the proper value to use, but climatic conditions divide the problem into two parts. In regions subjected to ice and snow the low coefficient of tires on ice will govern within a moderate speed range. In warm climates the effect of centrifugal force on driver and passengers will govern. These relationships can be expressed in terms of f. Professor Moyer of Iowa State College has done some excellent work in investigating skid characteristics of automobiles. He found that skidding on ice will occur at friction values ranging from 0.05 to 0.13. Thus for moderate speeds it would appear wise to limit f to 0.03 in regions subject to ice and snow in order to allow some factor of safety. As yet the top speed that a car can be operated under control on tangents on

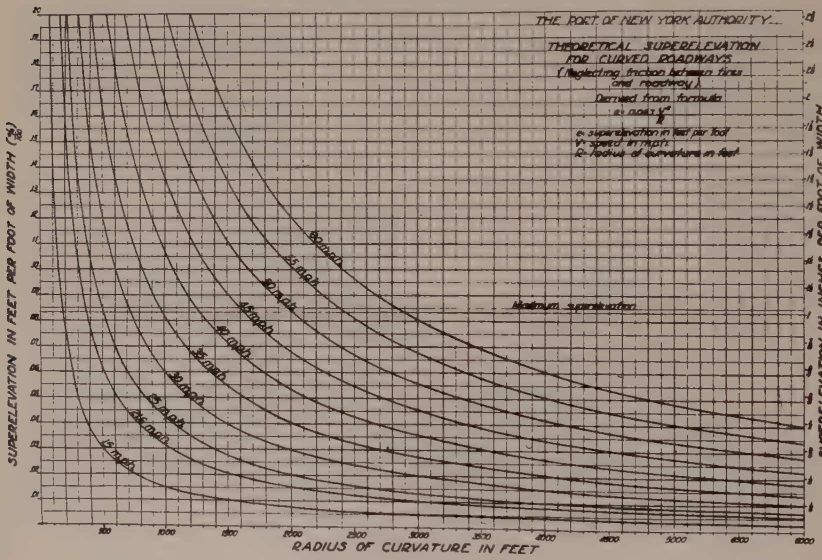


FIG. 2. — THEORETICAL SUPER-ELEVATION FOR CURVED ROADWAYS

ice has not been determined. It is, of course, unnecessary to bank curves based on icy conditions for speeds above that at which a car can be operated on ice on a tangent. Professor Moyer also found that when the side friction reached 0.20 the maximum speed had been attained for comfort of passengers and "balance" of the driver. It has been my experience that a certain amount of skill, not possessed by most drivers, is required to drive around a curve at a speed at which a coefficient of 0.20 is developed, and that a value of 0.10 is more likely to suit the majority of drivers. After a value of f is selected it is only necessary to

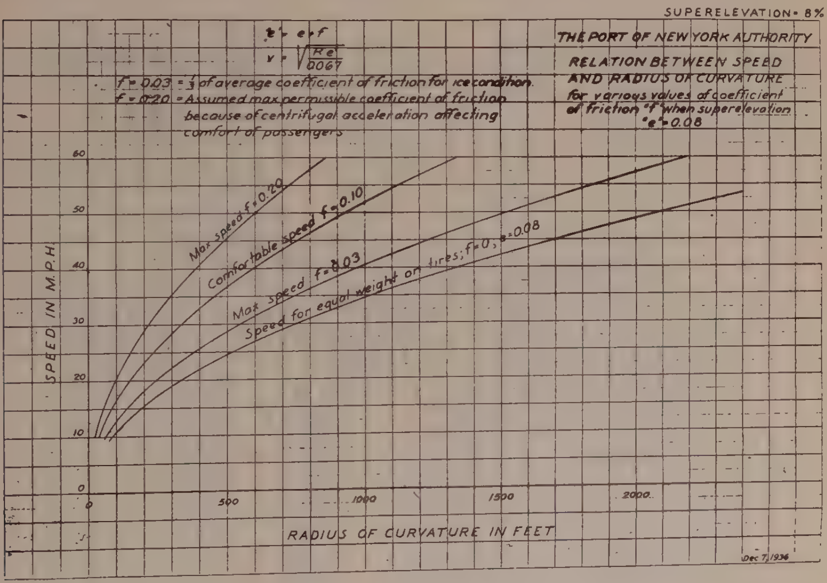


FIG. 3. — RELATION BETWEEN SPEED AND RADIUS OF CURVATURE

add that value to the maximum permissible super-elevation, and the relation of speed to radius can be taken from the diagram.

For convenience, a series of diagrams, each for a different rate of banking, can be produced showing the speed-radius relation for certain coefficients of friction. Fig. 3 shows such a diagram, with friction values of 0.20, 0.10, 0.03 and 0.0 for a bank of 8 per cent, or slightly less than 1 inch per foot. There will be noted for this rate of banking the speeds which fall below the curve of zero friction. Operating at speed-radii below the curve is uncomfortable and possibly dangerous, and for this reason it is desirable that the maximum bank be limited to approxi-

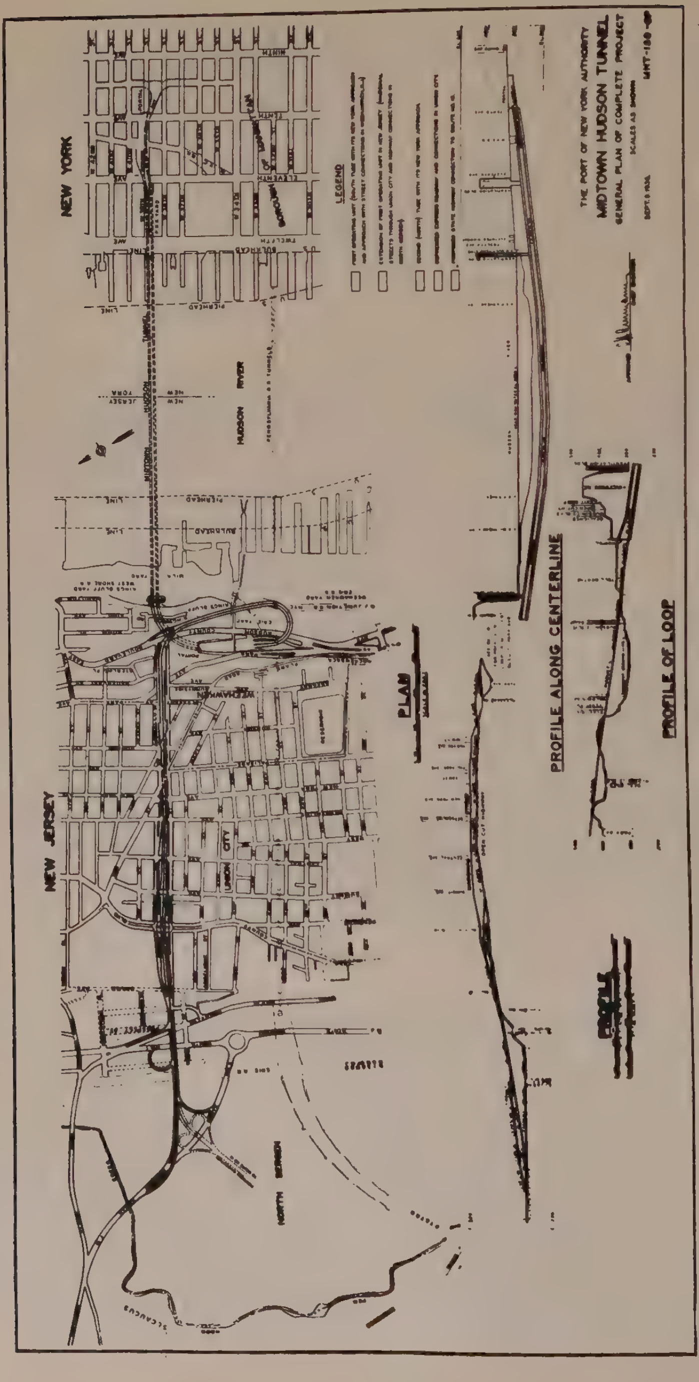


FIG. 4. — GENERAL PLAN OF MIDTOWN HUDSON TUNNEL

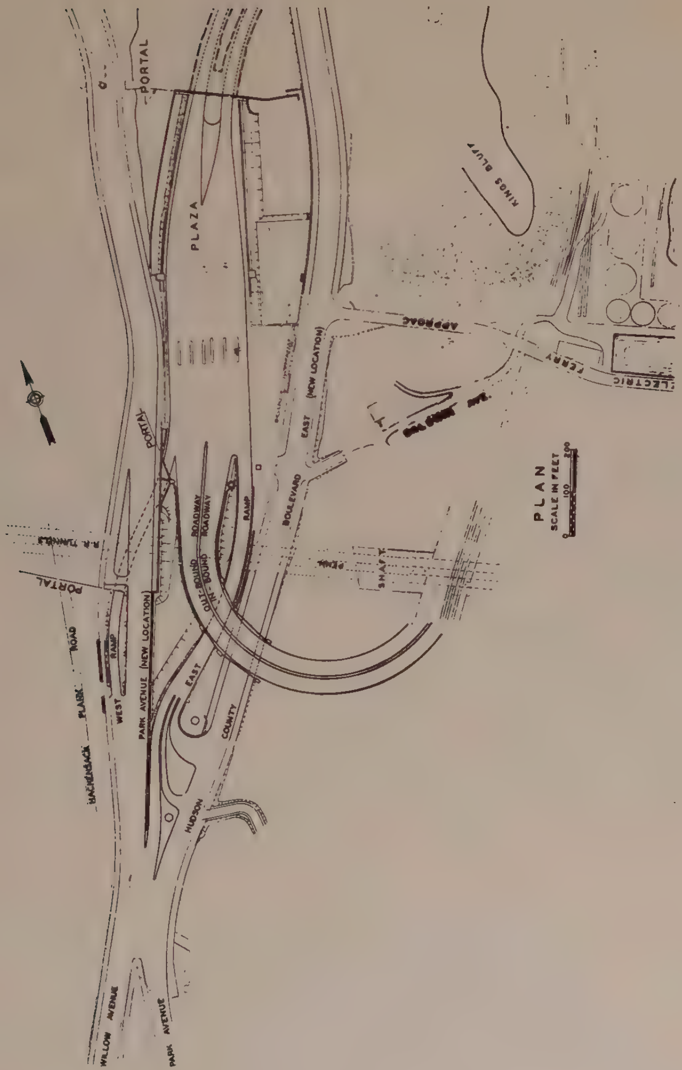


FIG. 5. — MIDTOWN HUDSON TUNNEL. LOCALITY PLAN AT NEW JERSEY PLAZA

mately 5 per cent (about one half inch per foot), and the radii correspondingly increased. This recommendation applies only to the cases where it is possible to increase the radii.

Before leaving the matter of curves and super-elevation it may be well to touch on transition spirals. Many engineers are prejudiced against their use, but there appears reason to believe that their use is essential where high-speed traffic is concerned. The reasons are quite clear when the *rate* of change of centrifugal acceleration from tangent to curve is investigated. The use of spirals also simplifies the problems connected with banking run-out.

It has been stated in structural circles that close attention to details is the secret of success in structural design. This is even more true in highway design, but as yet this fact has not been sufficiently appreciated as related to safety. Since recognition of accident hazards is largely brought about by a proper attitude of mind, and through certain processes of thought, it may prove of interest to note some of the lines of reasoning used in solving certain of the highway problems encountered in the design of the Midtown Hudson Tunnel which the Port of New York Authority is building at 39th Street, New York.

The general plan of the tunnel project is shown in Fig. 4.

The general conditions at the portion of the New Jersey plaza from the tunnel portals to the toll collection area are shown by Fig. 5, which is a photograph of a model of this entrance. The floodlights are located on towers which illuminate the entire area.

An idea of the scale of this plaza may be obtained by noting that the retaining wall is 65 feet high.

Fig. 6 shows the general outline of the New Jersey plaza. The size and shape of the plaza leading to and from the tunnels, and the gradual tapering at the portal end, are evident.

A low or depressed type of curb extends nearly to the portal. The space in the plaza is provided in order that caravans of trucks may accumulate. Careful thought was given to the danger of vehicles colliding with the portal face at the entering portal. On one side a full height curb was extended out from the face. On the other side a smooth, sloping surface of granite 2 feet 6 inches high was placed. If struck, there will be a tendency to slide along the face, with small damage to the vehicle. This device would not prove a safe solution where high speed is possible.

There were several problems at the portal which required attention. During initial operation traffic will operate in both directions, and thus the portal will be a location of greater hazard than elsewhere.

The change from artificial light to bright sun will cause impaired vision, and at the same time entering drivers will have to adjust themselves to the curve in the tunnel. To overcome the latter hazard the same radius of curve was carried out into the open plaza for such distance that entering drivers would become adjusted to the curve before the change in light. Near and at the portal the tunnel headroom was increased gradually in order that the light change will occur more slowly. The tunnel type of pavement was also carried out into the plaza some distance, to prevent the pavement change from distracting drivers.

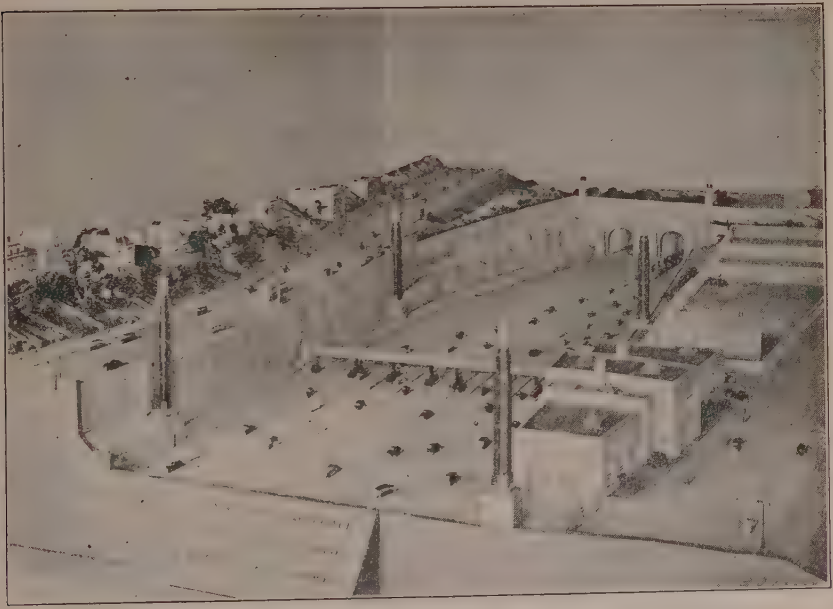


Fig. 6. — Model of New Jersey Approach, Midtown Hudson Tunnel

The banking at the portal was designed in such a manner as to tend to keep vehicles moving in opposite directions apart without volition of drivers. This was accomplished by providing different rates of banking for each lane. In the design, care had to be exercised not to reduce the rate of banking to too low a figure in the out-bound lane, because many times a vehicle will emerge from a dry non-skid tunnel on to ice, and it was essential that the lane be tilted up sufficiently to prevent side skid on ice. Fortunately, a rather close supervision of speed will be maintained in the tunnel. In all cases it has been felt that the beginning and ending of a curve constitute the most dangerous parts, and such differential lane banking has been generally employed in an effort to keep the vehicles apart. The concrete pavement slab lines have been arranged to act as traffic lines so far as possible.

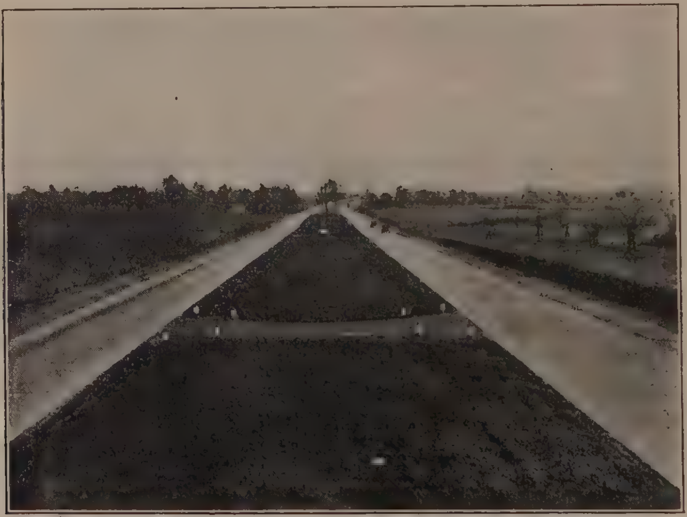
The distribution system of the approaches is shown in Fig. 6. The west ramp is off-bound, it passes under the street and emerges on the far side. The east ramp is an entrance ramp. The in-bound and out-bound roadways are the two main roadways, or what is known as the lower loop. They pass over the entrance ramp, over the new county boulevard, and out over a railroad yard. It may be noted that the roadways are stepped vertically in relation to each other. This was done as an expedient to obtain the necessary under-clearance for the ramp, and to ease the gradient of the inner roadway. Returning to the "off," or west, ramp it will be noted there is particular danger to drivers colliding with the end of the portal during periods when visibility is poor. Consequently the skew and the portal span were increased in order to locate the buttress at least 50 feet back from the "nose." This was done in spite of structural difficulties, which consisted of heavy dead and live loads and appreciable distortions due to temperature stresses. In order to further safeguard the public, a low mushroom light is placed just back of the "nose," and the face of the buttress will be flooded with light from one of the floodlights.

The approach to the banked sections of the main off-and-on bound roadways was given considerable study. It was necessary to provide ideal operating conditions while at the same time creating a pleasing appearance along all curb and wall lines. Particular attention is being given to producing a pleasing appearance in alignment and grades throughout the entire project. All curb "noses" opposed to traffic are rounded down to a height of 3 inches above the pavement.

The main approach roadways are laid out in a loop. The structure is elevated, the spans are quite long to clear railway tracks. The shaft to the Pennsylvania Railroad tunnels is adjacent to the railroad yard tracks. From a highway standpoint the conditions are critical. The loop, with a center line radius of 298 feet, occurs at the end of 1,100 feet of tangent on a 4 per cent descending gradient. Instead of using a tangent, however, a curve of 5,200-foot radius, 1,100 feet long, was inserted in the hope that the curvature, although slight, would tend to prevent drivers from descending at excessive speed. In order to soften the shock of entering the sharp curve some sort of spiral or transition was thought imperative. A quick search failed to reveal a true spiral which would permit an accurate determination of concentric curves and radial lines for the purpose of shop fabrication of steel, curbs, railings, etc., and therefore a series of compound curves totaling 225 feet in length, which simulated a true spiral, were employed. The curves compound from 5,200 feet to 4,000 feet, from 4,000 feet to 1,700 feet, from 1,700 feet to 500 feet, and lastly from 500 feet to 298 feet. This, it is realized, is a rather crude proceeding, but it is believed that operation will be satisfactory. Each roadway widens from 32½ feet to 36 feet, thus making it necessary to compute a separate set of compound curves for each marker line, as well as for the curbs and rails, in order that the lanes would widen uniformly and continuously. A great deal of care was taken in designing the banking run-out. Speed radius studies were made at 10-foot intervals for each lane, accurate profiles were drawn for the curb lines and for each lane marker line along the computed compound curves. After a number of trials the proper banking required for smooth operation was made compatible with good appearance. The total run-out length is 380 feet. It has been my observation that banking should start such a distance in advance of the curve that gravity will have sufficient time to act on the front wheels so that the car will tend to steer itself. A great deal of time is consumed in such detailed design, but it is believed to be worth while when it is remembered that approximately eight million cars will operate over this part of the approach each year for generations.

A common problem encountered in urban grade separations is the design of exit ramps. The problem often is complicated by having the exit on the outside of a curve, which introduces the hazard of locating the ramp "nose" directly in the path of cars traveling the main roadway. The solution most designers are tempted to use is to cut back and open up the entrance to the ramp with long radius curves, with the thought of reducing congestion at the ramp head. Such a procedure would make the "nose" of the ramp a hazard during adverse weather conditions. During fog and snow many drivers use the edge of the road as a guide, and in this case they would be lured into a position with the ramp "nose" directly ahead. If the motorist did not sense that the roadway curved he would be inclined to proceed straight ahead, possibly colliding with the wall back of the "nose." It is advisable to carry the main roadway curb as far along the curve as possible before reversing for the ramp. In one instance the curb ahead was set over or widened out 8 feet and the end of the wall set as far back as possible from the "nose" as well as widened.

At the present time much attention is being given divided or dual roadways for intercity congested routes. This is a decided step in the right direction, but unfortunately many of these designs contain details which are elements of danger, as, for example, light standards and public utility poles set near the pavement, poorly designed lighting, blind intersections, and the like. The general use of curbs constitutes a hazard and a prolific source of accident, but there are many who maintain they are safety devices. Originally curbing was used for three purposes, — to protect pedestrians, to form drainage channels, and to serve as a first line of warning and defence on bridges or high embankments. Today the curb is being employed to guide traffic and to prevent it from entering forbidden territory. The problem is divided into two divisions, — urban and open country locations.

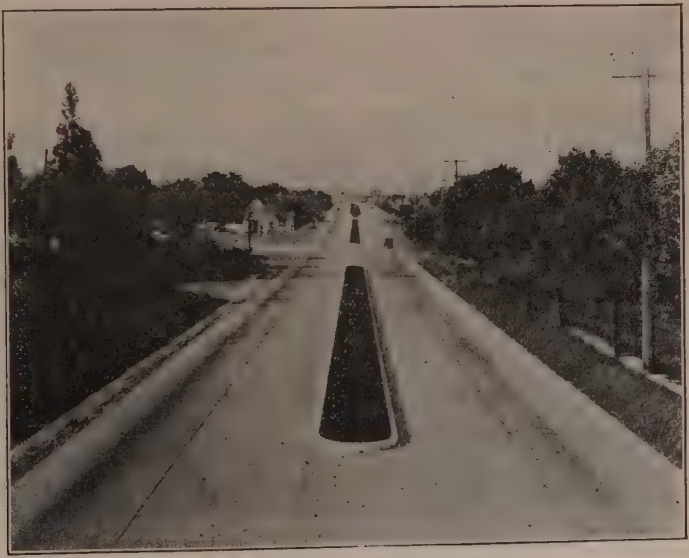


Courtesy of Delaware State Highway Department

FIG. 7. — EXAMPLE OF DIVIDED ROADWAY WITH WIDE CENTER RESERVATION

Fig. 7 shows a dual highway in Delaware through open country. The width between roadways is 50 feet. Note the absence of curbs and the wide, smooth shoulders. It is surprising the feeling of safety one has in driving over this highway. If an accident occurs directly ahead, or a car suddenly appears from the side, it is possible to dodge out on to the shoulder at high speed, or if the car goes into a skid it is possible to right the car and get back on to the pavement without striking some obstruction. Curiously enough, drivers keep to the right without supervision, and passing is no problem. The motorist realizes he is in no danger of being "squeezed," because it is always possible to swerve out on to the shoulder. Had curves been used the road would actually be less safe. The use of posts at the crossover is unfortunate, and is evidently an invention of the maintenance department.

Fig. 8 shows a treatment along the same highway through a town. The conditions here are somewhat different. Along the sides adjacent to the sidewalks, curbs are required for drainage and protection to pedestrians, but extra width must be provided to care for parking and to create a feeling of freedom. In the center the condition is much different. The purpose of the curb is to separate opposite direction traffic and prevent impatient motorists from using the space as a means of passing. There is a great difference of opinion as to the proper detailed design. This type has a corner beveled off at 45 degrees, but in spite of the bevel it is too high to be mounted in case of emergency. It is to be noted that the posted speed through the town is 25 miles per hour and speeds in excess of 35 or 40 miles per hour are very rare.



Courtesy of Delaware State Highway Department

FIG. 8. — EXAMPLE OF DIVIDED ROADWAY WITH MEDIUM CURB

Fig. 9 shows another type of separation curb more easily mounted than that shown in the previous figure; still, it is somewhat severe and may, if mounted at high speed, result in loss of control of the vehicle. These curbs were constructed with the use of white cement. Three weeks after being opened to traffic there was barely a section more than 50 feet long which did not show wheel marks where the curb had been mounted because vehicles had been "squeezed" or crowded. If the vehicles had not been able to dodge sideways a foot or so a sideswipe accident would have resulted. This section of highway carries an average daily traffic of 35,000. Traffic lights are so spaced that it is difficult to travel in excess of 35 miles per hour. At this speed the curbs can be mounted; but it is possible that at a speed of 50 to 60 miles per hour, mounting such curbs may prove more dangerous than a sideswipe accident. It is reported, also, that the earth grass plot is unsuitable because of softening during wet weather, such soft condition causing cars to overturn.

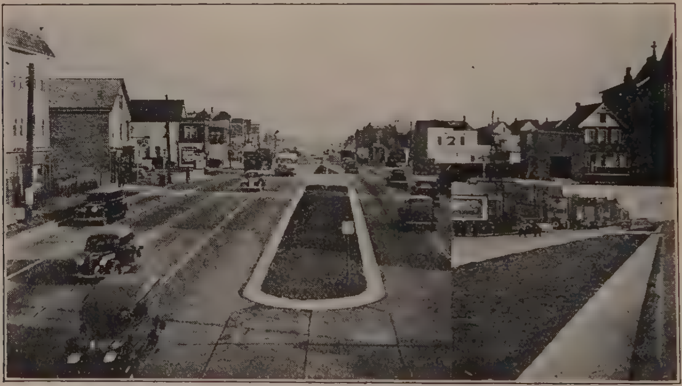


FIG. 9. — EXAMPLE OF MEDIUM CENTER STRIP WITH LOW CURB

An extreme form of separation curb design was found where the curb is 1 inch above the gutter and rises another 1 inch in a width of 12 inches. It is readily mounted at high speed, and it is quite possible that it may offer so little obstruction to mounting that drivers will run over it at will rather than use it only in an emergency. The reflector and sign have been destroyed many times during the past six years.

The New Jersey highway department is experimenting with a large variety of separation curb designs, and it is possible that definite recommended designs to suit the various different conditions will result from this experimentation. It would appear axiomatic that the separation space is no place for lighting standards, trees or any other form of obstruction.

This digression into a consideration of a few of the details which in the past have been considered as of no particular concern illustrates how some of these small matters may actually be the cause of accidents. Often the safer construction is less expensive than previously used designs. It should be constantly kept in mind that an automobile does not run on fixed rails and is thus not forcibly confined to its proper track. Therefore it is imperative that as much lateral "elbow" room, free from all obstructions, be provided to care for those emergencies occasioned by temporary loss of control or by crowding by other vehicles. On sober thought it is really remarkable that there are not more accidents when it is considered that every day twenty-six million separate individuals of various physical and mental capacities venture forth in a machine capable of terrific destructive force on to a highway system which is, in many ways, imperfect. If one be in a critical frame of mind and make a mental note of each detail which may prove a hazard to motor operation in darkness, fog, rain, snow and ice, he will be amazed at discovering the number of small things which are capable of producing accidents. A utility pole set near the edge of the street or highway is capable of wiping out an entire family.

The statement is made, with some truth, that the safest road is one which is a succession of sharp curves. On such a road high speed is physically out of the question if the curves are sharp enough. Because of the lower speed, accidents would not be so serious. The maximum in safety might be accomplished when a road is so bad or so congested that vehicles cannot operate faster than 15 miles per hour. Under such conditions accidents would be numerous, but few of them would be serious. But if we, as engineers, designed such roads our services would be dispensed with quite promptly. The public insists on expeditious road service. The public wishes to get to its destination promptly. It will not tolerate tortuous alignment or undue congestion which forces a slow rate of speed. In some localities vast sums have already been spent to give the public the type of service it demands, and yet, in spite of the large expenditure, many of the designs are imperfect. Many unnecessary hazards are being designed into projects which should be the last word in the highway engineer's art. This is not intended as a reflection on the designers, but only to show that our thinking has not been brought up to date. We are not yet thoroughly safety conscious.

Fig. 10 shows a highway which cost $4,850,000 per mile without right of way cost. It is over four miles long, carries a daily traffic of 42,000 vehicles, and no trucks are permitted. Note the light standards in the middle. Exit and entrance ramps are in the center. I understand the designers now regret this feature. The highway is posted for 35 miles per hour; yet in several places, due to sudden sharp curves and center ramps, it is posted for 20 miles per hour. The project was constructed in short sections, the first being opened nine years ago, the final is now being completed, while at the same time an extension is being started.

On another highway, about three and one half miles long, the cost was $19,500,000, including a right of way cost of $1,500,000. The average cost is $5,640,000 per mile. It was completed four years ago.

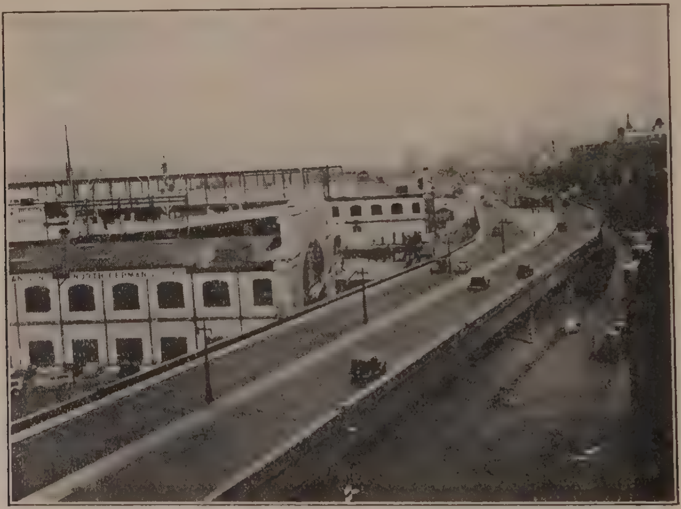


Photo by Savastano Studio

FIG. 10. — VIADUCT WITH CENTER DIVIDING STRIP; ALSO CENTER ENTRANCE RAMP (IN DISTANCE)

The accident record was so serious during the first year of operation that trucks were excluded from it. There is no separation between opposing lines of traffic, and there are center ramps. It is said that the designers saved a few tons of steel by the use of single center ramps instead of two side ramps.

In regard to these ramps, it has been learned that until quite recently the lighting effect at night was such that it was almost impossible to see the ramp head until quite near. A light was placed here which was of such a nature that the head was in shadow. This structure is a part of a trunk highway fifty-three miles long which carries an average daily traffic of 35,000 vehicles. Nearly all of it is at grade and consists of from four to six lanes without separation. The total project of fifty-three miles cost $59,000,000, including right of way. Within a year after its opening the public christened it "Death Highway." Six and seven car accidents were common. Yet this highway was designed by one of the most alert and competent highway organizations in the country, and was considered an outstanding example of modern highway design. During the past year a large portion has been made over into a divided way by jacking one slab over, constructing an additional lane, and installing separation islands with sloping curbs. This work is costing approximately $65,000 per mile.

In conclusion, I wish to emphasize that it has been my desire to present an idea for consideration, — the idea of safety consciousness, — rather than to advance definite recommendations. Highway designers must do all in their power to safeguard the public. Many times hazards are designed into a highway structure simply because the designer was intent on solving mathematical or structural problems, and therefore remained blind to the essential problem of providing a satisfactory structure based on its use by the public. Often it is no more expensive to design safely, but there are many times when safe design requires courage and conviction on the part of the designer. The highway should be considered as a unit and uniform standards applied, whether it be a tunnel or a bridge or it lies through the open countryside. As previously stated, it is manifestly absurd to construct all classes of roads to the same speed standards, and for this reason a searching highway transport analysis is required in order that the system can be properly classified, based on long-range planning. Fortunately, this phase is receiving recognition, and consequently designers should be ready with well-substantiated data in order that each class of highway may be designed for basic safety. But an awakened recognition of the problem is necessary. American engineering genius will solve the problem once the problem is thoroughly understood. It should, however, be remembered that in the past the error has been in setting the standards too low. We have purchased too cheap an article for real economy. There is real danger of a repetition of that error. It is quite possible that certain intercity trunk routes across the United States should be designed for a speed of 100 miles per hour. As citizens we should take the lead in establishing speed and driver control, but as designers we should face our particular problems with vision and courage.

Grateful acknowledgment is made to The Port of New York Authority and The Portland Cement Association for the preparation of slides, and to Messrs. E. B. Shrope and C. George Krueger of The Port of New York Authority staff for preparation of diagrams.

Discussion by Charles B. Breed,* Member, and Alexander J. Bone,† Member

General Discussion

Without question more attention must be given to safety factors in highway design than in the past, on account of *prevailing* high speeds. There is some question, however, whether the building of speedways for 80 to 100 miles per hour will solve the problem. Accident studies, including those conducted by the Massachusetts Highway Accident Survey, have shown that speed is the one outstanding contribution to accidents. Likewise, accident experience has shown that a large proportion of fatal accidents occur on straight, unobstructed roads where the highway is no factor. An increase in speed on straight, unobstructed highways will obviously result in more serious accidents. Roads for 80 miles per hour will encourage higher speeds than now prevail, and will result in more rather than less fatalities. Human frailty is the primary cause of most accidents; an instant's relaxation or distraction on the part of any driver under certain circumstances results in an accident — the faster the speed the more intent and unfailing must be the driver's skill and attention. Records also show that the vast majority of accidents occur to operators not previously involved in serious accident, indicating that judgment plays an important rôle. No doubt these drivers considered themselves safe drivers, but certain conditions arose which they were unable to meet and an accident resulted.

The ability of an individual to respond to an emergency is limited. Some people are faster observers and thinkers than others; some are more accurate observers and have better judgment than others. For every one there is a definite lag between perception and action. This lag, or reaction time, as it has been called, is of vital importance in driving a motor vehicle at high speed. It frequently determines the difference between a near accident and a fatal accident, escape or injury, life or death. Under actual driving conditions the reaction time is about one second (some people have shorter times, some longer). At 40 miles per hour a car will travel 57 feet in a second; at 60 miles per hour, it will travel 86 feet; at 80 miles per hour, 114 feet; at 100 miles per hour, 143 feet. At the higher speeds death will frequently overcome the victim before he can make a move to save himself. The energy to be dissipated when two vehicles strike, or when a vehicle strikes a fixed object, increases as the square of the speed. As the speed is increased, the likelihood of escape from death or injury in an accident is therefore greatly reduced.

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The highway engineer can do much to reduce the chance of accident by making existing highways fool-proof and by building future highways with a greater vision of their use. With this we are in full accord. The highway engineer cannot control the behavior of the driver, but he may make conditions more favorable for him in case he errs.

The immediate problem is critically to examine existing highways and correct those conditions which are unsafe. In other words, the hazards should be eliminated from existing highways so that they will be safe for reasonable speeds of 40 to 50 miles per hour. If a highway is, in the main, safe for 45 miles per hour but has a few curves that are only safe for 25 miles per hour, these should be flattened or superelevated, as the case may be. If sight distances are, in general, 500 feet on a highway, but at two or three places they are only 100 feet, these should be lengthened, and so on. The standard of a given highway should be maintained continuously from terminus to terminus, so that the driver will not come suddenly upon conditions requiring abrupt speed reduction and special driving effort unless he has adequate warnings to allow for a gradual reduction in speed.

The standards should apply uniformly day and night. All roads will not have the same "safe" speed; some will be 50 mile per hour roads, some 40 mile per hour, some 30 mile per hour (as in mountainous country). The speed should be posted, be reasonable, and *be enforced*. The present system of speed posting, where the signs read all the way from 20 to 45 miles per hour within a distance of half a mile, is one condition which should be corrected.

A standardization program of this kind is practical and clearly warranted. A considerable mileage could be standardized for the cost of one mile of a new 100 mile per hour road. Likewise, there are many streets in cities where as high an average speed as 10 miles per hour is impossible due to congestion. These should be given immediate attention. As motor vehicle use increases, congestion in cities will become more and more acute. Most of our state highways are crowded only a few days out of the year (summer Sundays and holidays), but the city streets teem with congestion 300 days a year. Here is where much of the gas tax is earned, but where very little of the gas tax money has been spent.

More money should now be spent for relief of traffic congestion in these metropolitan areas.

We do not believe that the "vast body of motor vehicle owners" want to drive 100 miles per hour. Perhaps we do not accurately sense the times. If they do want to travel at such speeds, they should not be allowed to do so for their own good. The sanitary engineer does not permit the public to drink polluted water, even though they are willing to take the chance. By the same token the highway engineer should not encourage the driving public to travel at 80 to 100 miles per hour when he is fully aware that technical knowledge, records of accidents and mental reaction measurements combined indicate clearly that sooner or later such speeds are suicidal for a fair proportion of the drivers who are reckless enough to travel at such speeds, as well as murderous to the innocent public who are willing to drive at not over 40 to 50 miles per hour. The highway engineer has a duty to society to protect it against its own ignorance. Few of the general public have any clear conception of the risks they run when operating at a speed of 60 to 80 miles an hour. Is it not our first duty to apprize the public of the facts we know to be true, and to prevent them, as far as possible, from attempting to drive trackless vehicles at the speed of the fastest railroad trains?

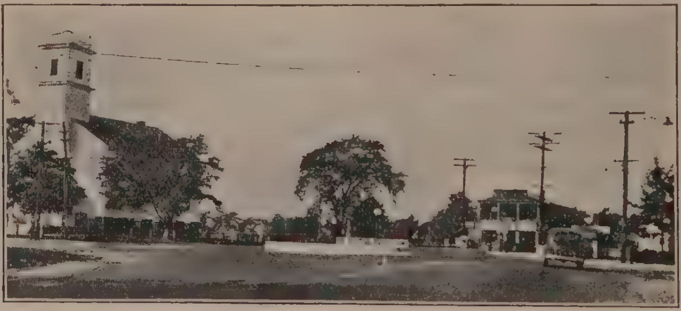

FIG. 1

Improvised traffic circle of small radius with high vertical curb. The curb bears the mark of many a collision. Note the white line which appears to guide traffic directly to the obstacle.

The highway engineer cannot be blamed too much for designing roads, built fifteen years ago, that are now obsolete. He could not foretell what the developments in vehicles would be, nor could the vehicle builder have predicted the improvements that have come in the last fifteen years. The highway engineer can, however, be criticized for allowing the danger spots to persist in the face of accident records — this has too often been done; and he should not be exonerated for designing modern highways which have immediately developed high accident records.

Makeshift traffic control devices often create a hazard worse than that which they are intended to correct. Fig. 1 shows an improvised rotary island which is too small to function as a traffic circle and merely invites accidents. The substantial concrete curb is covered with marks from cars that have collided with it.

Many of the safety devices designed into our highways recently have developed accident records that were not expected. These accidents, as a rule, are the result of drivers becoming confused and doing things they were not supposed to do. For example, trouble has developed at the entrance to ramps at grade separations, where a right turn is made in order ultimately to go left on the upper or lower cross street. The curves on these ramps are usually sharp and the entrance to them very abrupt. Rear end collisions have occurred when drivers have slowed down suddenly, trying to safely make the turn or read the signs. This difficulty may be solved by providing extra width in advance of these ramps so that drivers may slow down and make the turn from a lane outside (to right of) those provided for fast traffic. Another bad place is where traffic enters fast moving lanes from a ramp. Here, again, extra width should be provided so that cars can enter in a lane outside the fast lanes, accelerate, and then join with the fast main stream of traffic, at which point the normal width will be resumed. The present system is to install stop signs and pounce on all offenders. The Route 20–9 grade separation in Northborough, Massachusetts, is an example of an expensive layout which does not function well as a traffic control or safety device except by regulation and police enforcement. Fig. 2 shows a collision diagram for this grade separation. Note that nearly every accident has resulted from a wrong move on the part of some driver, and also observe the tendency of drivers to swing wide to the left before turning right into a ramp, with resulting collision with another vehicle passing straight through on their right. Why do they do this? Do they come on the ramp without warning, or is the radius of the ramp too sharp? Speed and confusion appear to be the principal factors in these accidents.

Part of the confusion at grade separations may be eliminated by more adequate markings and by providing separate approach lanes into and from the ramps. Simplicity must be adhered to in grade separation design, and warnings must be frequent and well in advance. A driver

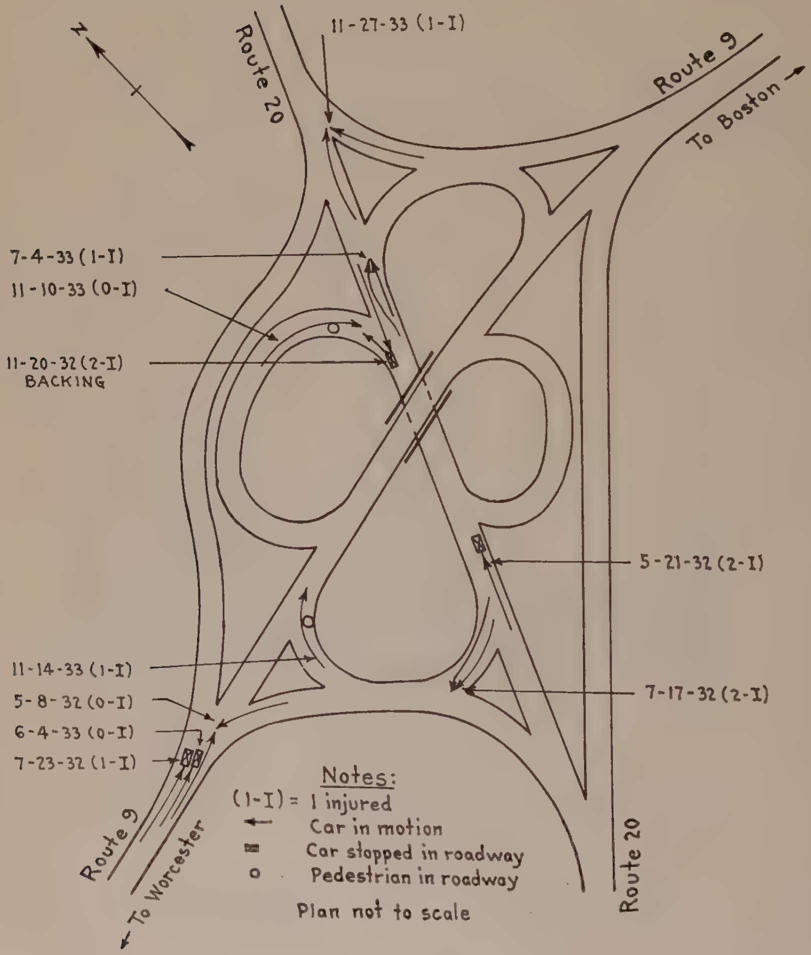

FIG. 2

Collision diagram, town of Northborough, Mass. Accidents 1930–33, inclusive, at intersection of Route Nos. 9 and 20. (From Massachusetts Highway Accident Survey.)

has no time to study the layout while he is approaching even at moderate speed.

The use of many signs on a single post at grade separations is not only useless but distracting. When traveling at even 30 miles per hour it is impossible to read more than one or two simple signs at once without too long a detraction from driving. The route numbers should be staggered, and presented to the driver one at a time well in advance.

Vertical curbs are frequently a hazard on rural highways. They are installed supposedly as a protection for pedestrians; but in rural locations where the pedestrian walks are located several feet away from the pavement and infrequently used, they have little safety value compared to the damage they can cause when a vehicle is forced off the pavement and runs into them. Their visibility is poor. It is almost impossible to see them at night; they appear to hide in their own shadow. It would be a definite advantage to drivers on many state roads if these vertical curbs were taken up and laid on their sides. This is particularly true of curbing on the center strip of a divided roadway. A sloping curb with white cement or white aggregate of some sort should be adopted as standard for edging on center reservations and on rural highways.

Mr. Noble emphasizes the use of a wide center reservation. The necessity for this is strikingly brought out by the inadequacy of the 10-foot reservation strip used on the Worcester Turnpike in Massachusetts.

Difficulty with this narrow center strip arises where it is carried across side streets and made to function as a rotary island. A driver entering from a side street and desiring to make a left turn must first make a right turn, travel with fast traffic to the first opening in the center strip and then make a U turn into the opposite stream of fast moving vehicles. In making this U turn, a narrow center strip provides no refuge against on-coming cars, and the driver must wait in the path of traffic for an opportunity to complete the turn. Since three quarters or more of the total roadway is required for this turn, a driver must wait for a completely clear road before he can proceed.

Figs. 3 and 4 illustrate the inadequacy of a 10-foot center strip for passenger cars, and for trucks, respectively, when attempting to make a U turn.

Fig. 5 shows an accident on the Worcester Turnpike in which the narrow dividing strip was a factor. This is grim evidence of the inadequacy of this narrow strip.

Figs. 6 and 7 illustrate the necessity for a minimum center width of 15 feet for a small passenger car and 30 feet for a large passenger

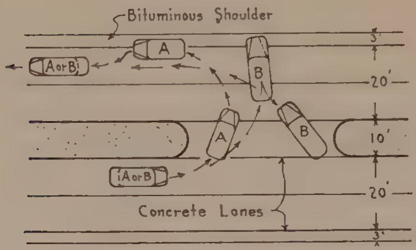

FIG. 3

Illustrating the inadequacy of a 10-foot center strip between two 20-foot roadways.

A is a small passenger car, 13 feet long and with a turning radius of 17 feet. This car must be driven on to the paved shoulder in order to complete a turn from the inside lane on one side to the outside lane on the other side.

B is a large passenger car, 18 feet long and with a turning radius of 25 feet. This car must be backed in order to make the turn.

The center strip is not wide enough to protect either car A or car B against the possibility of collision with cars traveling in the inside through lanes.

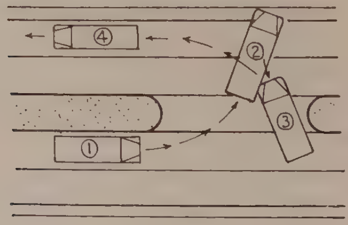

FIG. 4

Illustrating the inadequacy of a 10-foot center strip for a truck 25 feet long, and with a turning radius of 35 feet.

A truck of this size must be turned and backed as shown by positions (1), (2), (3) and (4). In this maneuver it completely blocks one roadway and half blocks the other.

car, respectively. If the same freedom of movement is allowed for a motor truck of moderate size a center width of 50 feet would be required, as illustrated in Fig. 8.

In many cases the cost of right-of-way wide enough to provide an ample center strip is prohibitive, and the inadequate 10-foot strip is the most that can be had. In this case it is doubtful whether the rotary movement through the center strip should be required of traffic. A straight crossing protected by stop and enter signs or signals would prove much less hazardous than a U turn in the path of fast traffic. In open country (where most high-speed roads should be located) the cost of a right-of-way 200 or 300 feet wide is not proportionally greater than for a 100-foot right-of-way. In this case sufficient width should be acquired to provide at least a 50-foot center strip.

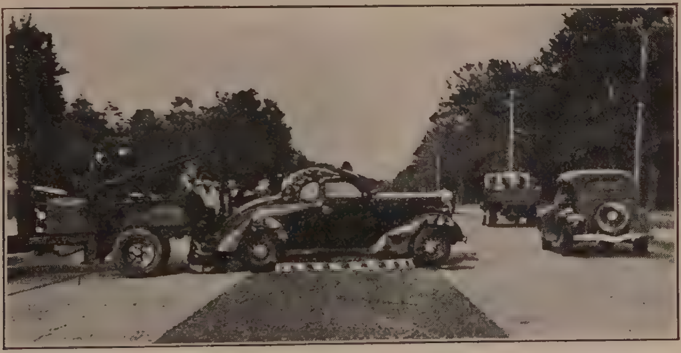

FIG. 5

Automobile accident on a divided roadway. The center reservation is 10 feet wide. Note how this small car overhangs the pavement on both sides of the center strip.

At best, crossovers are an undesirable feature in express highways, and their number should be kept to a minimum. Access to and egress from them should be allowed with the least possible interference with through traffic.

Fig. 9 shows a suggested layout for a crossover with deceleration and acceleration lanes. Two kinds of pavement are shown: the concrete pavement is provided for through traffic, and a non-skid bituminous type for deceleration, acceleration and turning lanes. A driver wishing to make a U turn, swings into the inside bituminous lane, slows down and turns through the opening and into the inside bitu-

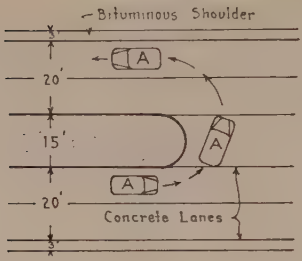

FIG. 6

Illustrating how a small car (A) with a turning radius of 17 feet requires a 15-foot center strip to make a turn from the inner lane on one roadway to the outer lane on the other, where the pavement is 20 feet wide on each side. There is just enough room between roadways for this small car (13 feet long) to stand free of traffic on the fast lanes.

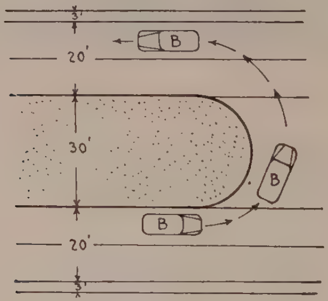

FIG. 7

Illustrating how a large car (B) with a 25-foot turning radius requires a 30-foot center strip to make a turn from the inner lane on one roadway to the outer lane on the other, where the pavement is 20 feet wide on each side. In this case there is ample room for the driver to wait in the crossover for a safe opportunity to complete the turn.

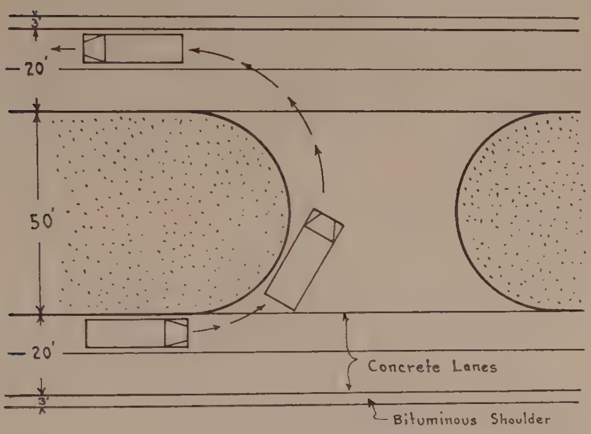

FIG. 8

Illustrating how a truck with a turning radius of 35 feet requires a center strip 50 feet wide in order to make a turn from the inner lane on one side to the outer lane on the other side.

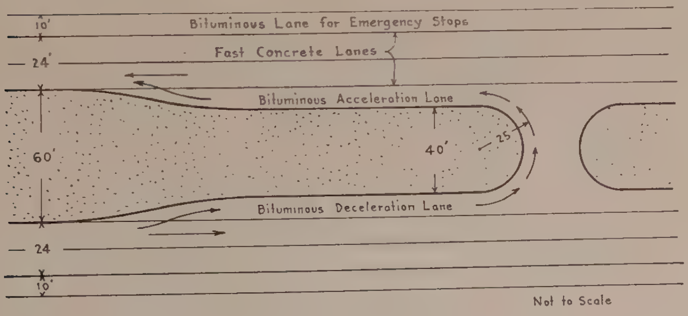
10'
Bituminous Lane for Emergency Stops
24'
Fast Concrete Lanes
Bituminous Acceleration Lane
60'
40'
25'
Bituminous Deceleration Lane
24
10'
Not to Scale

FIG. 9

A suggested design for a crossover in a divided express highway with deceleration and acceleration lanes for making turns into and from the crossover.

minous lane on the opposite side, picks up speed and joins with fast moving traffic on the concrete lanes. This allows for a continuous traffic movement at reduced speed. No difficulty should be encountered in entering the deceleration lane, but joining with the fast moving traffic at the end of the acceleration may prove more difficult, due to crowding from cars in the fast lanes. A stop and enter sign could be placed in the crossover so that entrance to the fast lanes would be made from a stop, and only at such a time when there was a break in the fast through traffic stream.

Economics of High-Speed Roads

High-speed highways will be so costly that they can only be considered for the more important intercity routes, such as between Boston and New York. Any one who has driven to New York will realize that no amount of remodeling will make the present highway suitable for speeds of 80 or 100 miles per hour. To provide such a speedway, another highway must be built on new location with all grade crossings eliminated. The cost of such a highway might run from $100,000 to $500,000 per mile, or somewhere in the vicinity of $60,000,000 for the total distance of about 200 miles between metropolitan areas. This is three to four times the total expenditures for state highway construction by the States of Massachusetts, Rhode Island and Connecticut in 1935.

The question arises as to who would benefit from such a road. The trucks and busses would surely benefit, but the trucks will be ruled off because they cannot maintain passenger car speeds and would interfere with the operation of passenger cars at 80 to 100 miles per hour. There would be little scenic value in such a road, for the driver at least; and not much recreational value except for those who crave excitement. Business men who travel in cars from city to city, and other drivers in a hurry to get from one region to another, would profit by the road. Much of the traffic on our present highways, however, is local traffic between cities and towns, and would not use the high speed road.

The only advantage of the high-speed highway is a saving in time, which is of real value to only a portion of the driving public. Should a large proportion of public funds be expended for the specific benefit of this particular class of drivers? Another justification for high-speed highways is military defence. This is obviously the major purpose of European high-speed highways, since highway traffic in Europe is very light compared to that in this country.

High-speed highways are also subject to the laws of diminishing returns. The faster the cars travel the farther apart they must be spaced and the less will be the ultimate capacity of the road in vehicles per hour. Fig. 10 illustrates theoretically the relative transportation service available from a lane of uninterrupted traffic at 45 miles per hour and 90 miles per hour, respectively. In return for the greater speed, vastly greater expenditure, greater risk of accident, and less capacity, a time saving of about two hours would be made in a trip from Boston to New York. The actual trip time would not be reduced 50 per cent, because the time required on congested highways to get from the end of the express road to the centers of the terminal cities must be added. Under favorable conditions, it is reasonable to assume that twenty minutes will be required in Boston and forty minutes in New York; then the total trip time is five and one half hours, and the saving in time would be about 40 per cent instead of 50 per cent. How much is this saving in time worth?

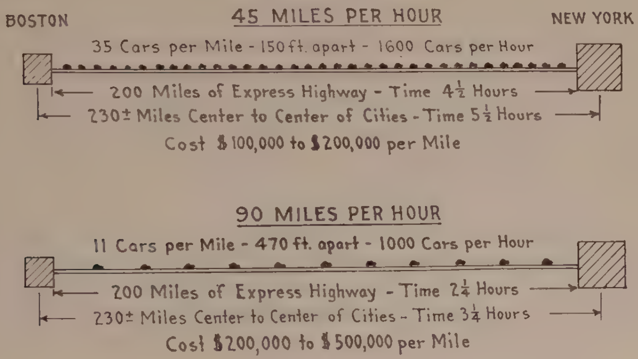
BOSTON
45 MILES PER HOUR
NEW YORK
35 Cars per Mile - 150 ft. apart - 1600 Cars per Hour
200 Miles of Express Highway - Time 4½ Hours
230± Miles Center to Center of Cities - Time 5½ Hours
Cost $100,000 to $200,000 per Mile
90 MILES PER HOUR
11 Cars per Mile - 470 ft. apart - 1000 Cars per Hour
200 Miles of Express Highway - Time 2¼ Hours
230± Miles Center to Center of Cities - Time 3¼ Hours
Cost $200,000 to $500,000 per Mile

FIG. 10

A theoretical comparison of the maximum transport service available from a single lane of uninterrupted roadway between Boston and New York at 45 miles per hour and 90 miles per hour respectively.

Spacings and cars per hour are from the following formula derived by the Massachusetts Highway Accident Survey, based upon the braking power of cars: $S=.0102M^{2.3}+1.47M+20$ where S is the minimum spacing of cars in feet at speed M in miles per hour.

In this connection it should be pointed out that there are already two fast means of transportation between Boston and New York, — the railroad, which is very safe and only takes four and one half to five hours from center to center of cities, and the airplane, which takes about one and one half hours between airports, and is becoming safer and safer each year. Why should there be duplication with public funds of fast means of transportation on a highway which is less safe than the means already existing by rail and by air?

Mr. Noble has stated that we are justified in spending 60 per cent more than we do now in order to build safe roads and eliminate accidents. This is based upon an estimate of economic loss from those accidents where the highway is a factor, which are considered to be 30 per cent of all accidents. Such a vast increase in highway expenditure would probably reduce the highway factor in accidents but would not proportionally decrease accidents as a whole, since the majority of accidents are traceable to causes other than the condition of the highway. More money should be spent on driver control and education rather than on expensive express highways which the average driver is not fit to use. The driver must be educated in the proper use of the highway. He must be made aware of his limitations and acquainted with the probabilities of the accidents. The present spirit of public indifference must be overcome.

Great concern is properly expressed for our soldiers lost in the World War, yet every year a like number of our citizens are killed as a result of highway accidents, and a greater number injured. The Battle of Gettysburg was one of the most sanguinary events in history, yet the number killed in that four-day battle was no more than are killed in three months on our highways. In the four years of the Civil War, about 110,000 men were killed, which is about the same number that are killed every three years on our highways.

Not until sudden death and maimed relatives visit our homes a little more frequently will public sentiment fully revolt against this ghastly spectacle. This time is fast approaching. Now is our chance to give an everlasting contribution to humanity, not by encouraging the construction of highways for greater speed, but by educating the public in the limitations of the average human mind, in the many and subtle dangers in motor vehicle operations, that speeds of 40 to 50 miles per hour should be the maximum on any highway for reasonable safety both for the driver and for the other fellow. We have an abiding faith that the sanity of the average citizen will eventually prevail when he fully realizes the hazards of high-speed highway travel.

RECENT EXAMPLES OF THE DESIGN OF ALUMINUM ALLOY STRUCTURES

By B. J. Fletcher *

(Presented at a meeting of the Designers Section of the Boston Society of Civil Engineers held on March 10, 1937)

For the year 1906 the world production of aluminum was 16,000 tons. According to present estimates, the world production of aluminum for 1936 was more than 300,000 tons. In many fields this rapid growth has been due to factors such as attractive appearance, resistance to corrosion, high electrical and thermal conductivity, and workability. In other fields, including structures and construction equipment, the increasing use of aluminum and its alloys is due primarily to the combination of low specific gravity and high strength. Where weight reduction means decreased initial or operating costs or increased capacity for either stationary or moving structures, aluminum alloys merit the consideration of the designing engineer.

An efficient design combines functional adequacy with economy of material. The designer must find and follow this narrow path between inadequacy and waste. Strength and durability consistent with the service requirements of a structure are always essential. Elimination of waste is imperative in the design of aluminum alloy structures where excess material exacts a double penalty in the form of added first cost and a reduction in the desired weight saving.

To prepare an efficient design an engineer must be familiar with the service to which the structure will be subjected and with the manner in which his materials will behave under the conditions imposed by that service. The mechanical properties of the aluminum alloys used for structural purposes have been investigated thoroughly and are given in specifications published by several technical societies, government agencies and manufacturers. The special and routine tests made on aluminum alloy products assure uniform quality under definite material specifications.

A simple tabulation of ultimate tensile strength, yield strength, elongation and other mechanical properties has a limited value in present-

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ing a true picture of the behavior of structural members fabricated from the material in question. These data supply a foundation from which to work, but they give only a hint of the form of the structure which should be erected on that foundation.

Specifications for the design of aluminum alloy structures have been prepared which give the unit stresses and the proper proportions for members fabricated from commercially available commodities. Such specifications limit the working stresses in columns, beams and tension members of a typical aluminum alloy structure. Those who have worked on design problems involving the use of aluminum alloys have found such specifications helpful but inadequate, because there is no such thing as a typical aluminum alloy structure. Where these alloys have been used for structural applications some special problem is invariably involved. Existing specifications for the design of steel buildings, concrete highway bridges or steel railway bridges cover applications where tremendous amounts of material are used in structures subjected to similar service requirements over a large part of the country. No skyscrapers, highway bridges or railway bridges exist in which aluminum alloys form the principal material of construction. These alloys have been used for bridge floors in Pittsburgh, Pennsylvania, and Bridgeport, Connecticut; for emergency bulkheads for dams in West Virginia, Ohio and Pennsylvania; for dragline booms in Louisiana, Pennsylvania, Alaska, Greece, South America and other parts of the world; for power shovel dippers in strip coal mines throughout the Middle West; and in many other applications, each highly specialized as to service conditions and economic requirements. A general design specification must sacrifice maximum economy in order to assure safety and durability under all conditions existing throughout a wide field. In aluminum alloy structures such a compromise is economically impossible, since waste of material in any individual structure increases first cost and defeats the ever-present objective of maximum weight saving.

Aluminum alloys are not a material for the handbook engineer who, all too frequently, makes a superficial study of the loads to which the members of a structure will be subjected and then selects his angles or beams or columns from a set of tables. Aluminum alloys do offer a flexible and economical material of construction to the designer possessed of the ability and experience necessary to make a careful analysis of a specific problem, to determine the factor of safety essential to that application, and to select the proper materials to suit his needs. To aid such engineers a new type of specification has been prepared. This general specification presents information on the aluminum alloys available commercially, together with discussions, formulæ and curves giving the ultimate strength of a variety of structural members fabricated from these alloys. From this basic information a competent engineer can prepare detailed specifications, in terms of allowable working stresses, for the use of the designer. By this means adequate safety and elimination of waste will be assured by adapting each major aluminum alloy structure to the particular ends which it must serve. This specification will be available in the near future.

It is of importance to turn from general considerations of design to actual examples of the work which has been accomplished in the field of civil engineering with aluminum alloys. These examples include power shovel dippers, dragline booms and emergency bulkheads for dams. The question has been raised as to whether a dragline boom or a power shovel dipper is a structure or a machine part. Such classification is unnecessary in view of the fact that manufacturers of this equipment have learned that an application of the methods of analysis and design familiar to all structural engineers saves money and improves their product. These particular examples have been selected because they represent a wide variation in the range of exactitude with which imposed loadings and resultant stresses can be determined. They typify three different methods of approach to designs embodying the use of aluminum alloys.

During the past few years manufacturers of many types of construction equipment have been confronted with the problem of reducing the dead weight of their products. Truck bodies, material handling skips, and excavating buckets are characteristic of this group. Conventional materials, service requirements and factors of safety vary over a wide range for such equipment, but in each case the design and the service are such that loads and stresses are highly indeterminate. In each case an inexperienced or careless operator can wreck the equipment. An abstract knowledge of materials and rational processes of design cannot be combined to produce a satisfactory unit unless the designer has a background of actual experience with the performance of similar units. A blacksmith who has spent years repairing mine skips can build a durable and economical unit, where an engineer, thoroughly familiar with the properties of materials and methods of structural analysis, would have little chance of designing a satisfactory skip unless he had a background of practical experience in actual mining. In work of this type it is usually possible to get the blacksmith and the engineer together to produce a job better than either could achieve alone.

An outstanding example of the application of aluminum alloys to equipment subjected to severe and indeterminate working conditions is furnished by the large power shovel dippers which are used for strip mining. A consideration of the operating cycle of a 16 cubic yard dipper gives some indication of the variety and severity of the service encountered. At the start of its stroke the dipper teeth are thrust forward into the strata of dirt or rock which lie above the coal seam. As the dipper is forced up through the bank it tears through alternate layers of hard and soft material. Large rocks may fall from high up on a 40-foot bank into the dipper or on to its sides or bail. It wrenches free from the bank with a full load and starts to swing to the spoil bank. The shovel swings, first accelerating and then retarding the bucket with its 25-ton load. Finally, the operator trips the door latch, the door swings open and the load pours out. The door swings back, forced by the dropping load, and may strike the bumper block on the stick with a blow that can be heard a mile away. Frequently a boulder has lodged on the teeth and falls, striking the door as it swings shut. Then the bucket swings back to the pit to begin the cycle again. This complete operation of digging, swinging, dumping and swinging back has taken less than a minute. Twenty-five tons of earth and rock have been moved over 200 feet and lifted 50 feet. This goes on twenty-four hours a day, six or seven days a week. It would take a supreme optimist to attempt to reduce the wrenching and shocks of that cycle to numerical terms of pounds or tons of load on dipper parts, and a second optimist of equal caliber to attempt to reduce those loads to terms of stresses in shell plates, back, door and bail.

For years power shovel dippers were made of massive steel castings and plate, and they weighed over 2 tons per yard of dipper capacity. This imposed a severe limit to the size of dippers which could be handled. In the building of the Panama Canal the 5 cubic yard power shovel dippers which were used were considered enormous. Today an aluminized power shovel dipper, built by the Marion Steam Shovel Company and operated by the Northern Illinois Coal Corporation at Wilmington, Illinois, has a capacity of 32 cubic yards. During 1936 it moved 6,300,000 cubic yards of earth. The main strength members of this dipper are aluminum alloy castings and plate. The dipper weighs slightly over 2,000 pounds per cubic yard of capacity.

The design of this dipper was not a problem in stress analysis. It was developed from the contribution of an intimate knowledge of working requirements and service conditions by the operator, a knowledge of power shovel design and construction based on fifty years of experience by the manufacturer, and a knowledge of the characteristics of aluminum alloys by the producer of the castings and plate.

The first large aluminized stripping dipper was a 16 cubic yard unit built in 1934 and still operating. The design was based on experience with a 12 cubic yard steel dipper which it replaced, giving a 33 per cent increase in daily output. In this dipper the bail, shell plates and door plates were the heat-treated aluminum alloy 27S–T, which weighs 0.101 of a pound per cubic inch, and has an ultimate tensile strength of about 60,000 pounds per square inch, a yield strength of about 43,000 pounds per square inch and an elongation of about 12 per cent in 2 inches. The lip, teeth and bottom bands are manganese steel, which has excellent resistance to the wear caused by highly abrasive rock and sand which is handled. The body consists of two 27S–T plates which are separated in the back by cast spacers to give a box girder construction which takes the thrust of the stick. The main shell is still in service after two and one half years.

Back connection pads are castings in alloy 220–T4, which has the following typical properties for standard individually cast test bars: weight, 0.092 of a pound per cubic inch; ultimate tensile strength, 44,000 pounds per square inch; yield strength, 26,000 pounds per square inch; elongation, 11 per cent in 2 inches.

It is a recognized fact that the strength of castings depends on the foundry methods employed as well as on metal quality, as shown by test bars. The massive door hinge castings, shaped like a fishhook with a varying I form of cross section, presented a real problem in foundry technique. During the experimental period the proper locations of gates, risers and chills were worked out. Sequences of pouring operations and of heat-treatment methods were studied. Castings were X-rayed and then tested under operating conditions. This work resulted in a steady improvement of metal quality. At the same time conventional hinge designs were modified to give proper stress distribution and to eliminate points of stress concentration on the complicated sections. The actual stresses caused by huge rocks falling from the teeth on to the door plate, or by the slamming shut of a 3-ton door, may never be known, but study and analysis have resulted in the proper distribution of metal and the proper methods for producing castings of a high degree of uniformity. The fact that there has been no hinge trouble of any kind on these dippers for more than a year indicates the effectiveness of this approach to the design problem.

The bail of a dipper is subject to reasonably exact analysis. The maximum pull on the bail is known because, if the dipper teeth encounter excessive resistance in passing up through the bank, the electric hoist motors will stall. Impact effects are decreased by the flexibility of the long-hoist cables. Taking the stall load on the hoist lines plus a moderate impact factor as the maximum load, the bail is analyzed as a curved beam, allowing suitable factors for stress concentrations due to curvature, and considering the ends of the beam to be either fixed or free. Several forged bails have been designed by this method and have proved uniformly satisfactory in service. Bails for stripping and coal-loading dippers have been forged from aluminum alloy 14S–T. Forgings made from this alloy have shown the following mechanical properties: weight, 0.101 of a pound per cubic inch; ultimate tensile strength, 65,000 pounds per square inch; yield strength, 50,000 pounds

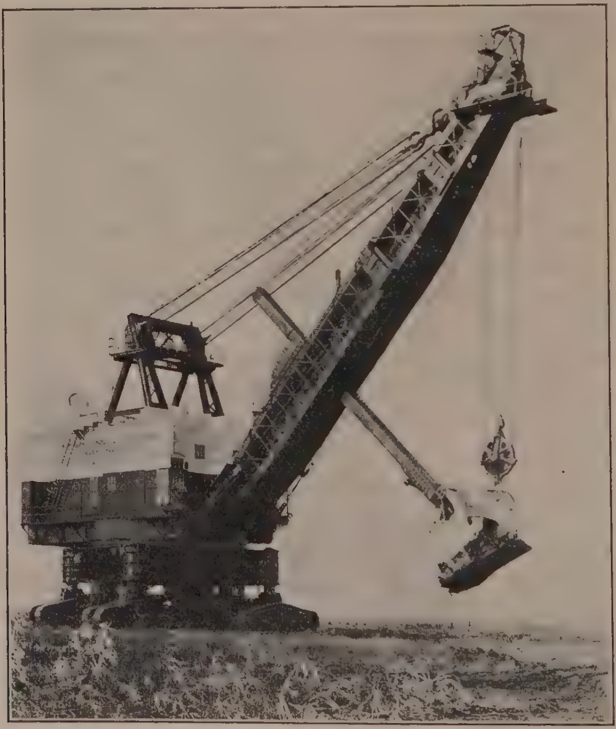

FIG. 1. — ELECTRIC POWER SHOVEL EQUIPPED WITH 32 CUBIC YARD ALUMINIZED DIPPER

This, the world's largest, power shovel moves overburden in an Illinois strip coal mine.

per square inch; elongation, 10 per cent in 2 inches. The forgings have a rectangular cross section of varying depth. The bail for a 10 cubic yard dipper weighs 1,320 pounds in aluminum, and is the largest forging ever made in an aluminum alloy. For the 16 and 17 cubic yard dippers forged bails could not be used, as size was limited by existing forging equipment, so built-up bails of a box section were used. In such bails efficiency is reduced by the extent and complexity of the connections necessary to make all parts of the assembly work as a unit. The success of forged bails has emphasized the fundamental design fact that homogeneity of section and freedom from stress raisers are of the first importance.

As more dippers were built and operating experience mounted, designs were steadily improved. The 32 cubic yard dipper has been in service for sixteen months and has handled nearly 8,000,000 cubic yards of dirt. In that time there has been no replacement or failure of any aluminum alloy part.

The design of power shovel dippers is an art based on cut and dried methods, but even here structural analysis has proved of the greatest value in the perfection of certain details.

In the field of construction equipment types of service vary widely in severity and determinacy. The second example chosen is excavator booms. A dragline or dredge boom does not come in contact with the material handled, except in the inadvertent, and fortunately rare, case of collapse. The weight of the bucket and earth load is known within reasonable limits, and the forces due to swing accelerations can be estimated. Knowledge on these points is incomplete and inaccurate, but still a fair approximation of loads and stresses can be made for a specific case. The first dragline boom in which an extensive use was made of aluminum alloys was a 75-foot boom handling 1½ cubic yards. It was designed and built by the Bucyrus-Erie Company in 1930, and functioned well and efficiently in the anthracite coal fields until it was finally lost in April, 1935, when the whole machine was buried by a landslide.

In building the Mississippi River levee system, which has given such a good account of itself in the recent flood, the basic problem was the movement of large quantities of earth over comparatively short distances. One of the most efficient machines used was the dragline excavator. These machines travel on rails, caterpillars or tubs. They work in soft, mucky ground with insecure footing where stability is important. For efficiency, boom lengths and bucket capacities must be a maximum. A light-weight boom permits a long reach and preserves machine stability. Through the use of aluminum alloys, boom structure weights are decreased about 50 per cent, and reach is increased 15 or 20 per cent. This cuts down rehandling of dirt and will increase construction speed, using a given machine, from 15 to 25 per cent under favorable conditions of steady operation.

A dragline boom is essentially a strut subjected to heavy compression loads due to the weight of the bucket and dirt acting through hoist lines and boom support guys. In addition to this axial compression the boom is subjected to vertical loads because of its own dead weight and to a horizontal load at the outer end of the boom due to swing acceleration and deceleration. Where loads are known it is a relatively simple matter to arrive at the loads and stresses in the various members of this latticed strut.

The material used for these booms is aluminum alloy 17S–T, which has the following typical physical properties: weight, 0.101 of a pound per cubic inch; ultimate tensile strength, 58,000 pounds per square inch; yield strength, 35,000 pounds per square inch; elongation, 20 per cent in 2 inches. In common with all of the alloys of aluminum, 17S–T has a modulus of elasticity of 10,300,000 pounds per square inch, — approximately one third that of structural steel. This factor is of primary importance in the design of such structures as dragline booms, where heavy compression loads are encountered in the main chord members. If steel and aluminum beams of identical size are subjected to equal loads, the deflection will be approximately three times as great for aluminum as for steel. Similarly, an aluminum column has a greater tendency to bow under load than a steel column of equal cross section and length. Since the stiffness of a member varies directly with the moment of inertia as well as with the modulus of elasticity, a relatively small increase in the dimensions of the cross section insures adequate stiffness. The virtues of bulk are too often overlooked in the attempt to save weight in structures. Aluminum alloys are the only commercially available structural metal in which weight saving can be effected without sacrificing the benefits of bulk of section which are so essential to simplicity of fabrication and to the stability of both individual members and of the structure as a whole. Through a moderate increase in the depth and width of the boom as a whole, together with a slight increase in the cross section of the individual members, ample stability and strength are attained, together with a weight saving of about 50 per cent in the boom structure.

The design of dragline booms is developed directly from a study of loading conditions and not from a comparison with former steel booms.

To determine the stress distribution in the members of built-up aluminum struts sixteen scale models have been built and tested by the Aluminum Research Laboratory under the direction of R. L. Templin and E. C. Hartmann. The models were tested as columns subjected to axially and eccentrically applied loads, as beams, and as struts subjected to torsion. Strain gauge readings were taken to determine average stresses, and the stress distribution on the cross section of members and around riveted joints. These model studies have assisted in selecting efficient forms, arrangement of members, and details for dragline booms. As is often the case, this research has contributed even more to the general knowledge of structural design than to the limited field of dragline booms.

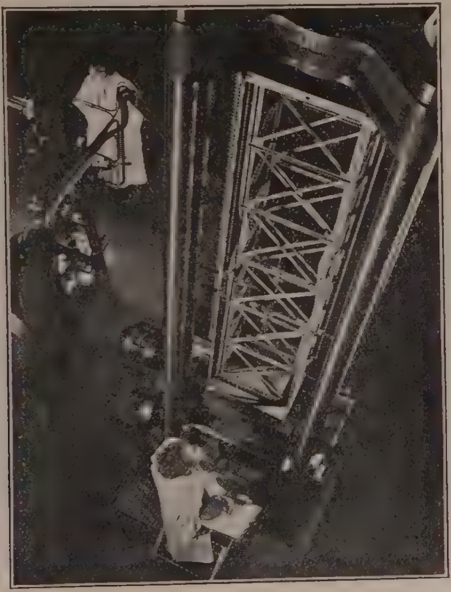

FIG. 2. — TESTING A MODEL OF AN ALUMINUM BOOM SECTION

Extensive laboratory and field tests form a background for the design of aluminum alloy structures.

While most aluminum dragline booms have proved durable and economical, the performance of certain booms indicated that design assumptions were in error. Almost no exact knowledge existed as to the magnitude or nature of swing loads on a working boom. Laboratory tests could not duplicate actual field operating conditions, so strain measurements were made on a full-sized boom working in the field. Those familiar with the conventional strain gauge will appreciate that no one could ride a vibrating, whipping boom and secure accurate readings. The gauges used were of the self-recording type known as Scratch Strain Gauges. With these gauges changes in length of a 3-inch gauge line are recorded as scratches on a chromium plated brass target. This record is photographed and enlarged one hundred times or more, and strains are scaled from the curves. The gauges are only about 4 inches long and are so light that they can be fastened to any structural member. A combination of gauges gives the distribution of stress at a given cross section, and from this the average stress and the load on the member can be computed.

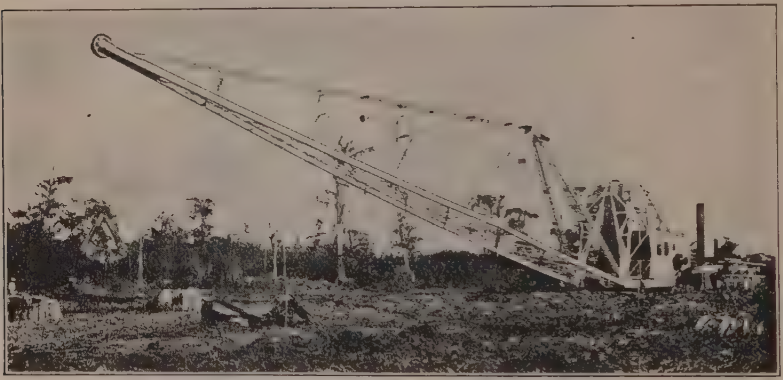

FIG. 3. — LEVEE BUILDERS ON THE MISSISSIPPI RIVER SYSTEM USE LONG ALUMINIZED BOOMS

This 240-foot dredge boom uses a 150-foot aluminum alloy peak section to reduce dead weight and increase working range.

From these field tests it was found in one case that stresses at a certain point varied from 26,900 pounds per square inch compression to 5,000 pounds per square inch tension as the machine operated through a digging cycle. This was considerably in excess of calculated values based on the loadings originally assumed. A study of the data indicated that excessive horizontal bending loads were caused by jerky, rather than uniform, swing acceleration due to rough acting clutches. The data also indicated that the fundamental methods of design and analysis were correct, provided the proper acceleration factor was used. Thus through a combination of laboratory and field tests the durability and economy of such structures have been materially improved.

Last year an unusual type of boom was built for a dredge now operating in the Atchafalaya Floodway in Louisiana. This boom has a total length of 240 feet, of which the outer 150 feet are aluminum alloy. The design includes relatively shallow plate girders for the bottom chords which carry the compression loads. This bottom chord is made to work with the boom support cables by a system of aluminum posts and steel cables. This design is proving most stable and satisfactory in service. It handles a 6 cubic yard steel grab bucket with a loaded weight of 30,000 pounds.

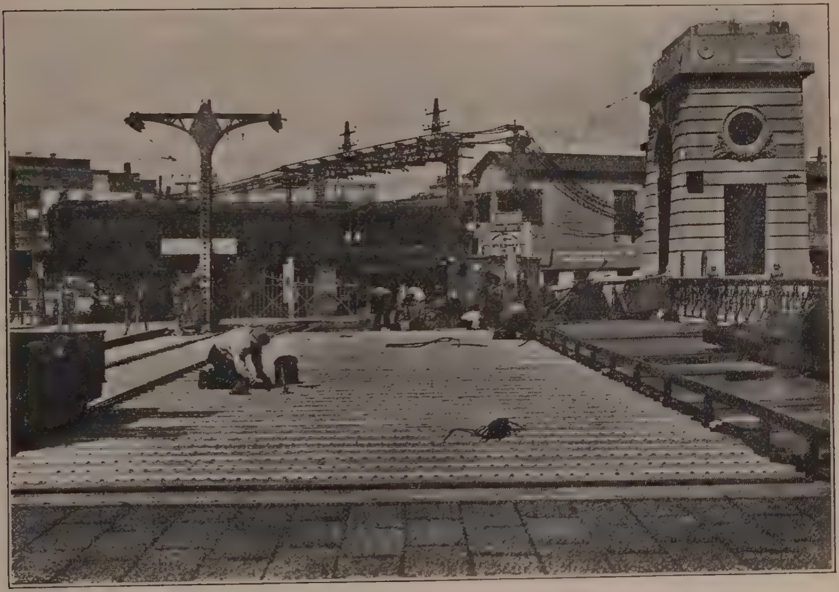

FIG. 4. — CONSTRUCTION OF ALUMINUM ALLOY FLOOR FOR STRATFORD AVENUE BASCULE BRIDGE AT BRIDGEPORT, CONNECTICUT

Large, light-weight shop-fabricated floor panels speed erection and cut dead weight. Aluminum alloy panels surfaced with mineralized asphalt blocks provide a light-weight floor and a surface adapted to all traffic requirements and to protection of bridge structure.

The experience with aluminum booms shows that careful analysis based on laboratory and field tests will give efficient designs even where conditions of service are severe and loads vary over a wide range.

In contrast to power shovel dippers and dragline booms, where the knowledge concerning loads and stresses is limited, are the more familiar types of structures, such as the bridge floor installed on the old Smithfield Street bridge in Pittsburgh in the fall of 1933. The light-weight aluminum alloy highway and railway floor system eliminated the necessity of building an entirely new bridge, and gave the structure, which was originally built in 1883, a new lease on life.

The advantages of light-weight flooring are obvious on a bascule type bridge, and led to the installation last spring of an aluminum alloy floor on the Stratford Avenue bridge on the Boston Post Road at Bridgeport, Connecticut.

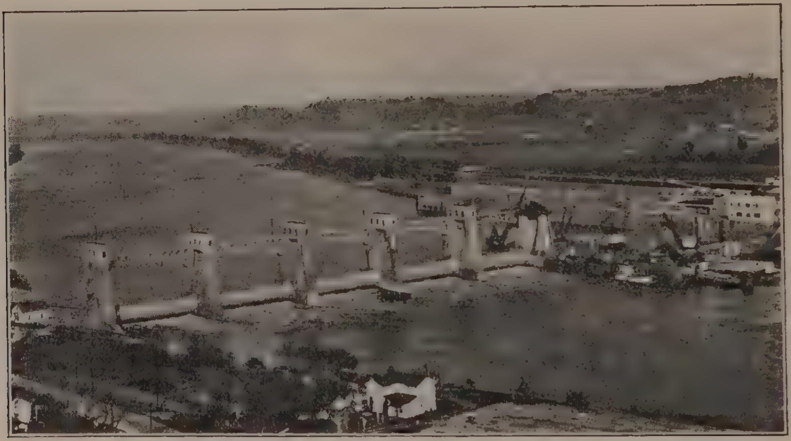

FIG. 5. — GALLIPOLIS DAM ON OHIO RIVER, UNDER CONSTRUCTION

Aluminum alloy emergency bulkheads are placed by a derrick boat to close openings when steel roller gates must be raised for maintenance or repair.

Although there may be some doubt as to the maximum loads to which the trucks will subject our bridge floors in the future, the weight of water will never change, and bulkheads designed to resist a known head of water furnish an excellent example of a design problem where loading conditions are accurately known and where a refinement of stress analysis is fully justified.

The Gallipolis Dam on the Ohio River is typical of the structures which the United States Engineer Corps is building to improve navigation on the Mississippi River system. In movable-crest dams of this type the level of the upstream pool is controlled by huge steel roller gates. At Gallipolis these cylindrical gates have an over-all length of nearly 130 feet and a diameter of 22 feet. The effective height of the gate is increased to 29 feet by a projecting flap which seats against the sill of the dam. The level of the pool above the dam must be maintained at all times, so a method is provided for closing the opening when a roller gate is lifted for painting or repairs. The emergency bulkheads used to close one gate opening consist of seven aluminum alloy units, each having an over-all length of 129 feet 6¾ inches, and a depth per unit of 4 feet 4 inches. Each unit has two parallel trusses which are 13 feet 10¼ inches back to back of main chords. The bottom chords are connected by a water-tight skinplate. When in use the seven units are stacked one on top of the other, with the trusses lying horizontally upstream from the skinplates. As any one of the seven units may be on the bottom, all of the units are identical. The load on the bottom unit, 4 feet 4 inches high, may reach 7,410 pounds per lineal foot, or 945,000 pounds per unit. It will be appreciated that a gate with a span of 130 feet designed to support a 500-ton load must be a bulky structure.

When the United States Engineer Corps designed these bulkheads in structural steel they found that each unit would weigh about 78 tons. To handle such a heavy load it would be necessary to use a large crane running on a heavy bridge from pier to pier across the full 1,100-foot length of the dam. By using nickel steel for the bulkheads it was estimated that the weight per unit could be reduced to 44 tons, but still a heavy bridge and crane would be required. A design was prepared in which aluminum alloy 27S–T was specified. The estimated weight of each aluminum unit was 28 tons. This weight could be handled by a derrick boat operating upstream from the dam, thus eliminating the crane and permitting the use of a light bridge between piers. Through the use of aluminum alloy bulkheads the entire design of the dam was changed. Long gate openings offering the minimum restriction to river flow were made possible, and the expense of a heavy crane and bridge were eliminated. In the design of these bulkheads a basic working stress in tension of 22,000 pounds per square inch was used. The specifications required that the 27S–T shapes and plate should meet the following minimum values:

Ultimate tensile strength (pounds per square inch)	58,000
Yield strength (pounds per square inch)	45,000
Elongation in 2 inches (per cent)	8

Before the final decision to use aluminum alloy bulkheads was made, a test unit approximately 104 feet long by 4 feet high was fabricated from 27S–T and actually tried out at the Marmet Dam on the Kanawha River in West Virginia. This dam was built several years ago and is equipped with the heavy construction crane and bridge necessary to handle steel bulkheads. The test of the aluminum unit demonstrated that it had adequate strength and would handle and seat properly under any conditions of flow which would be encountered in practice. This unit will be used at the Winfield Dam on the Kanawha River in West Virginia.

The design of the Gallipolis bulkheads was based on the results of this test, together with a careful study of the service conditions and of the characteristics of aluminum alloy 27S–T. Chord members consist of web plates and flange angles. Economy of material was effected through continuity of joints in compression members, obtained by using gusset plates inserted between the flange angles. Both gusset plates and web plates had edges milled for bearing. The question of deflection of the units under water loads was solved by providing an upstream camber of about 4 inches, and by an ingenious arrangement of end rollers set in housings which permit them to swivel about a vertical axis.

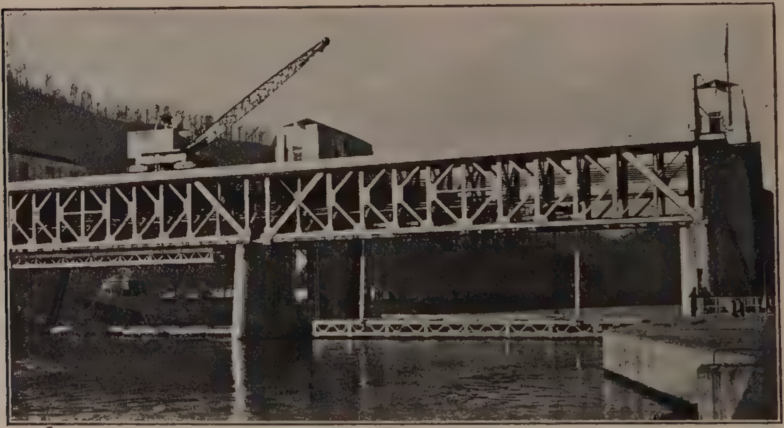

Fig. 6. — Test of Aluminum Alloy Emergency Bulkhead at Marmet Dam, Kanawha River

The use of light-weight emergency bulkheads for recent dams will eliminate heavy bridge and crane shown here.

The design of any structure is affected by the methods of fabrication which must be employed. In the case of the emergency bulkheads for the Gallipolis Dam, fabrication was carried out in the shops of the Nashville

Bridge Company. This shop had never built an aluminum alloy structure before; but, in spite of this, highly satisfactory results were obtained, due to intelligent effort on the part of the designer and fabricator. Standard steel working tools were used for shearing, punching, reaming, milling and riveting. Alloy 27S–T owes its high strength to heat treatment, and for this reason welding and flame-cutting operations which would anneal the material are not permitted. Where coping cuts were required the work was done on a band saw similar to the type employed for hardwoods. Cold driven annealed steel rivets were used in the fabrication of built-up members. Twenty-six thousand five hundred steel rivets, three quarters inch and seven eighths inch in diameters, were driven cold, using an 80-ton squeeze riveter with closely controlled air pressure. Driven heads were of a flat cone type. The use of cold riveting permits a very close clearance of rivets in the holes, which were subpunched and reamed to the nominal rivet diameter. These rivets were of good appearance and exceptionally tight. This process of cold riveting appears to be gaining in favor and has good possibilities for strength

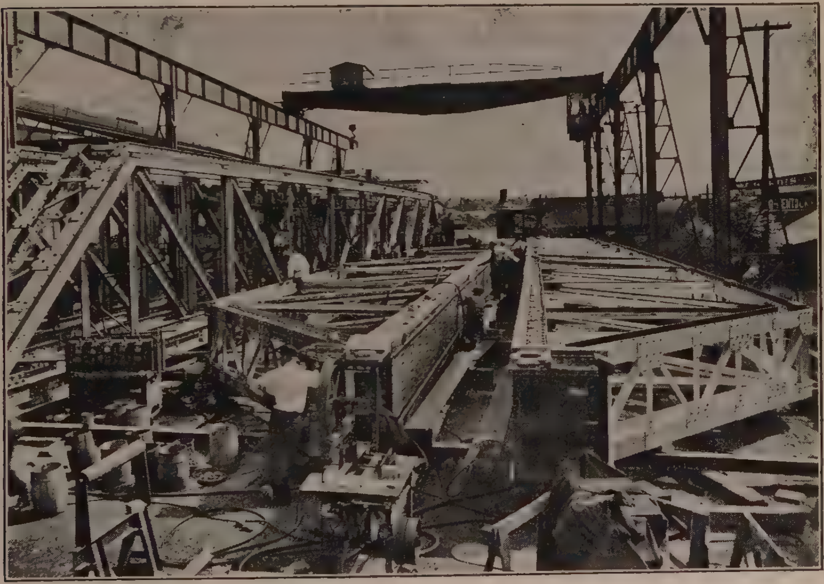

FIG. 7.—ALUMINUM ALLOY EMERGENCY BULKHEADS FOR GALLIPOLIS DAM.

Standard fabricating tools and methods are used in the construction of aluminum alloy structures.

and economy. Rivets in the assembled units were hot driven, using standard steel rivets and pneumatic hammers.

The final step in the fabrication of the bulkheads was the caulking of skinplate joints with pneumatic tools, attachment of oak bearing timbers to act as seals on the upstream face, and painting with a priming coat of zinc chromate paint followed by two coats of aluminum paint.

Designs and specifications for the emergency bulkheads for the Gallipolis Dam were prepared by the United States Engineer Office, Huntington, West Virginia, under the direction of Lieut-Col. John F. Conklin. They are an excellent example of the weight-saving and resulting economies which can be attained with aluminum alloys through the use of efficient design methods where loading conditions are definitely known.

At present a set of similar aluminum alloy bulkheads is being built for the Winfield Dam on the Kanawha River, and a third set of bulkheads will be built this spring for the Emsworth Dam on the Ohio River.

This brief sketch gives some idea of the manner in which design problems involving aluminum alloys have been approached in the past. There is a good back-log of practical experience at present, and this is constantly being increased by work in fabricating shop, design office and the research laboratory. Engineers interested in the possibilities of weight reduction through the use of aluminum alloys are assured of complete co-operation in the solution of their problems.

OF GENERAL INTEREST

PRIZES AWARDED AT ANNUAL MEETING ON MARCH 17, 1937

Special Boston Society of Civil Engineers Prize

TO PROF. JOHN B. BABCOCK, 3D, MEMBER

Presentation made by President H. K. Barrows

President H. K. Barrows stated that the Presidential Address delivered a year ago has been of particular interest to members of the Society. It is therefore appropriate that the Board of Government adopted the recommendation of the Committee on Prize Awards, namely, that a Special Prize be given to Prof. John B. Babcock, 3d, Past President of the Society, for his admirable historical and biographical address entitled "The Boston Society of Civil Engineers and its Founder Members," presented at the annual meeting, March 18, 1936. President Barrows, with great personal pleasure, presented the prize, which consisted of books: "Fifty Years of Boston," a memorial volume issued in commemoration of the Tercentenary of 1930; "Elements of Railroad Engineering," by W. G. Raymond, revised by Riggs and Sadler; and "Early History of Modern Civil Engineering," by Kirby and Lawson.

Professor Babcock, in accepting the prize, expressed his pleasure not only in being the recipient, but also referred to his enjoyment and interest in the research made in the preparation of the address.

The Desmond FitzGerald Medal

TO PROF. ALBERT HAERTLEIN, MEMBER

Presentation made by Ralph W. Horne, Chairman of Committee on Award

The Desmond FitzGerald Medal was instituted and endowed in 1910 by the late Desmond FitzGerald, a Past President and honorary member of this Society, and is awarded annually for a paper presented to the Society by a member, and published during the year, which is adjudged worthy of special commendation for its merit. The Committee on Award this year, consisting of Frank A. Marston, Lewis E. Moore and Ralph W. Horne, selected from the list of a number of excellent papers one which was recommended to the Board of

Government for the award. The paper selected was entitled "The Design of Statically Indeterminate Trusses," by Prof. Albert Haertlein, member, presented at a meeting of the Designers Section held on January 8, 1936, and published in the JOURNAL of the Society, April, 1936. On behalf of the Board of Government Mr. Horne presented the Desmond FitzGerald Medal to Professor Haertlein.

Professor Haertlein expressed his appreciation of the honor bestowed upon him by the presentation of this medal.

The Sanitary Section Prize

TO ALMON L. FALES, MEMBER

Presentation by Ralph W. Horne, Chairman of Committee on Prize Awards

On behalf of the Board of Government, Past President Ralph W. Horne, Chairman of the Committee on Prize Awards, presented the prize to Almon L. Fales, member, for his paper entitled "Chemical Precipitation of Sewage and Industrial Wastes," read at a meeting of the Sanitary Section held on April 1, 1936, and published in the October, 1936, JOURNAL. The prize consisted of books: "Civil Engineers Handbook," by Merriman, and "Handbook of Chemistry and Physics," edited by Charles D. Hodgman.

Mr. Fales accepted the prize with appropriate remarks.

The Designers Section Prize

TO HERMAN G. DRESSER, MEMBER

Presentation by Ralph W. Horne, Chairman of Committee on Prize Awards

On behalf of the Board of Government, Past President Ralph W. Horne, Chairman of the Committee on Prize Awards, presented the prize to Herman G. Dresser, member, for his paper entitled "Some Features of Sewer and Culvert Design," read at a meeting of the Designers Section held on March 11, 1936, and published in the October, 1936, JOURNAL. The prize consisted of books: "Rahmenformeln," by Kleinlogel; "Continuous Frames of Reinforced Concrete," by Cross and Morgan; and "Vibration Prevention in Engineering," by Arthur L. Kimball.

Mr. Dresser accepted the prize with appropriate remarks.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

JANUARY 27, 1937. — A regular meeting of the Boston Society of Civil Engineers was held this evening at North Hall, Walker Memorial, Massachusetts Institute of Technology, and was called to order by the President, H. K. Barrows, at 7 P.M. Seventy-five members and guests were present. About 50 persons attended the supper in the cafeteria.

The President announced the deaths of the following members:

Philip D. Borden, elected a member March 15, 1882, died December 24, 1936; Edmund S. Davis, elected a member December 18, 1878, died December 17, 1936; Luther Dean, elected a member June 17, 1891, died January 9, 1937.

The Secretary reported on the election of new members, on January 27, 1937, as follows:

Grade of Junior: John B. Ledden.

Grade of Student: John N. Squarebrigs.

The President stated that at the previous meeting of the Society, on December 16, 1936, a vote was passed regarding the use of the current income of the Permanent Fund. A similar vote, final action, is necessary at this meeting.

Voted, That the Board of Government be authorized to use the current income of the Permanent Fund in so far as necessary for current expenses of the Society.

The President announced that the February meeting will be held on Friday the 19th, at the New Lecture Hall, Harvard University.

Mr. A. B. Edwards, reporting for the B. S. C. E. Committee on Membership and Publicity, outlined the drive for membership in this Society and urged members to invite engineers to join.

The President stated that this meeting is a joint meeting with the Highway Section of B. S. C. E. and with the Engineering Societies of New England. He therefore requested Prof. Henry B. Alvord, Chairman of the Highway Section, to take charge of the meeting. Professor Alvord then introduced the speaker of the evening, Mr. Charles M. Noble, of the staff of the Port of New York Authority, who gave an interesting and illustrated talk on "The Factor of Safety in Highway Design."

This paper outlined the fundamental problems facing highway engineers today because of the rapid increase in speed of modern automobiles. In view of the fact that the pioneering era of highway development has closed, more searching processes of thought will be required in the further advance of highway design. Some of the problems of design of the Midtown Hudson Tunnel and approaches being constructed by the Port of New York Authority were discussed.

Brief discussion of the paper was given by Mr. Maxwell Halsey of Harvard University, Prof. H. B. Breed, Prof. A. J. Bone of Massachusetts Institute of Technology, Edgar F. Copell, Traffic Engineer, Massachusetts Department of Public Works, and others.

The meeting adjourned at 9.45 P.M.

EVERETT N. HUTCHINS, *Secretary.*

FEBRUARY 19, 1937. — A regular meeting of the Boston Society of Civil Engineers was held this evening at the New Lecture Hall, Harvard University, Cambridge, and was called to order by the President, H. K. Barrows. This was a joint meeting with the Northeastern Section of the American Society of Civil Engineers and the Student Chapters, A. S. C. E., at Harvard, Massachusetts Institute of Technology and Tufts College, and the Northeastern University

Section of B. S. C. E. The meeting was attended by 950 members and guests. About 100 persons attended the dinner at the Harvard Faculty Club previous to the meeting.

The following was elected to membership at the meeting of the Board of Government today:

Grade of Student: John C. Finn.

The President introduced the speaker of the evening, Mr. Charles F. Goodrich, Chief Engineer of the American Bridge Company, who gave a very interesting talk on "The San Francisco–Oakland Bay Bridge." This bridge, opened to traffic on November 12, 1936, is the greatest bridge in the world for several reasons: its cost, $77,000,000; its total length, 8¼ miles, 4½ miles of which are over water; its 51 piers, six of which are deeper than any other piers in the world; its tunnel through Yerba Buena Island, with the largest bore in the world (52 feet by 65 feet); and its 200,000 tons of steel and 1,000,000 cubic yards of concrete.

The bridge includes the only large twin suspension bridge ever built, and the longest and heaviest cantilever bridge in the United States and third longest in the world. It is a double-deck structure providing for six lanes of pleasure automobiles on the upper roadway and three lanes on the lower roadway. There is enough wire in the cables of the suspension bridge to reach three times around the earth. It is estimated that 52,000,000 persons will cross this bridge in 1937.

The talk was illustrated with sound motion pictures describing the building of this great structure.

The meeting adjourned at 10 P.M.

JOHN B. BABCOCK, *Secretary pro tem.*

MARCH 17, 1937. — The eighty-ninth annual meeting of the Boston Society of Civil Engineers was held today at the Boston Chamber of Commerce, and was called to order at 5 P.M. by the President, H. K. Barrows.

The minutes of all previous meetings of the year which have been printed in the various issues of the JOURNAL were approved as printed.

The Secretary reported the election of the following new members:

Grade of Member: Milton J. Adams, John S. Brinkler, Timothy F. Creedon,* John P. Cronin, Philip Dixon, John L. Elliott, Emil A. Gramstorff, Henry A. Hamilton,* Carl A. Held, Richard W. Johnson, Francis M. Mahard, A. Albert Minichiello,* William T. Morrissey, Bernard J. Parker, Herman C. Protze, Herman J. Shea, Harald M. Westergaard.

Grade of Junior: William A. Grady,† Wilfrid C. Pitts.†

Grade of Associate: Stuart Huckins.

Grade of Student: Charles H. Fish.

The annual reports of the Board of Government, Treasurer, Secretary and Auditors were presented. Reports were also made by the following committees: Social Activities, Relation of Sections to Main Society, Welfare, Membership and Publicity, Library, Run-off (final report), 1936 Floods, and John R. Freeman Fund.

Voted, That the reports be accepted with thanks and placed on file, and that they be printed in the April, 1937, JOURNAL.

Voted, That the incoming Board of Government be authorized to appoint such committees as it deems desirable.

The report of the Tellers of Election, Walton H. Sears and Edward S. Averell, was presented, and in accordance therewith the President declared the following had been elected officers for the ensuing year:

President — Arthur D. Weston.
Vice-President (for two years) — Prof. Gordon M. Fair.
Secretary — Everett N. Hutchins.
Treasurer — Charles R. Main.
Directors (for two years) — Carroll A. Farwell, Howard M. Turner.
Members of Nominating Committee (for two years) — George C. Houser, Henry B. Alvord, Albert E. Kleinert.

The retiring President then delivered his annual address on "A Study of the Effect of Temperature upon Different Reactions and Processes."

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* Transfer from Grade of Junior.

† Transfer from Grade of Student.

Sixty-five members attended this part of the meeting.

The meeting adjourned to assemble at 7.30 P.M., the annual dinner being held during the interim. A musical entertainment was given during the dinner period.

The President then called the meeting to order for the presentation of prizes.

The President, referring to the exceptionally fine biographical and historical paper on "The Boston Society of Civil Engineers and its Founder Members," given as the Presidential Address at the annual meeting, March 18, 1936, by Prof. John B. Babcock, 3d, presented a special prize to the author. The prize consisted of books: "Fifty Years of Boston," a memorial volume issued in commemoration of the Tercentenary of 1930; "Early History of Modern Civil Engineering," by Kirby and Lawson; and "Elements of Railroad Engineering," by W. G. Raymond, revised by Riggs and Sadler.

The President then called upon Past President Ralph W. Horne, Chairman of the Committee on Award of the Desmond FitzGerald Medal, to present this prize to Prof. Albert Haertlein, member, for his paper on "The Design of Statically Indeterminate Trusses," presented at a meeting of the Designers Section on January 8, 1936, and published in the JOURNAL for April, 1936.

Also Mr. Horne presented the Sanitary Section prize to Almon L. Fales, member, for his paper on "Chemical Precipitation of Industrial Wastes," presented at a meeting of the Sanitary Section on April 1, 1936, and published in the October, 1936, JOURNAL. The prize consisted of books: "Civil Engineers Handbook," by Merriman and Wiggin, and "Handbook of Chemistry and Physics," edited by Charles D. Hodgman, M.S.

Also Mr. Horne presented the Designers Section prize to Herman G. Dresser, member, for his paper on "Some Features of Sewer and Culvert Design," given at a meeting of the Designers Section on March 11, 1936, and published in the October, 1936, JOURNAL. The prize consisted of books: "Rahmenformeln," by Kleinlogel; "Continuous Frames of Reinforced Concrete," by Cross and Morgan; and "Vibration Prevention in Engineering," by Arthur L. Kimball.

President Barrows stated that the newly elected President, Arthur D. Weston, is recuperating from an illness and was unable to attend this meeting. His mention of the sterling qualities of the new President occasioned a hearty and long applause.

The President introduced the speaker of the evening, Lieutenant-Commander C. F. Grisham, United States Submarine Base, New London, Connecticut, who gave an extremely thrilling and interesting account of his personal experiences as a part of his talk on "The Narrative of the Loss of the United States Submarine S-5 off the Delaware Coast, September 1, 1920."

Ninety-one members and guests attended the dinner.

The meeting adjourned at 9.15 P.M.

EVERETT N. HUTCHINS, *Secretary.*

Sanitary Section

FEBRUARY 17, 1937. — A special meeting of the Sanitary Section of the Boston Society of Civil Engineers was held Wednesday, February 17, 1937. Preceding the meeting, 25 members assembled at Patten's Restaurant, Court Street, Boston, for dinner.

The meeting was called to order by the Chairman, Paul Howard, at 7.15 P.M. in the Society rooms, with 51 members and guests present. A nominating committee was chosen for the selection of officers to be elected at the annual meeting. Samuel M. Ellsworth presented a paper on "Water Sanitation and the Planning Movement," in which he outlined the principles of national, state and municipal planning organizations and their relation to sanitary engineering. An interesting discussion followed.

The meeting adjourned at 8.15 P.M.

RALPH M. SOULE, *Clerk.*

MARCH 3, 1937. — The annual meeting of the Sanitary Section of the Boston Society of Civil Engineers was held March 3, 1937. Preceding the meeting 23 members gathered at Patten's Restaurant for dinner.

The meeting was called to order by Chairman Howard at 7 P.M. in the Society Rooms, with 69 members and guests present.

The report of the Executive Committee for the past year was read and accepted.

The report of the Nominating Committee presenting nominations for officers for the coming year was submitted, as follows:

Chairman — Richard S. Holmgren.
Vice-Chairman — Samuel M. Ellsworth.
Clerk — Ralph M. Soule.
Executive Committee — Paul F. Howard, Gail P. Edwards, Lester C. Hollis.

No further nominations were made, and it was —

Voted, That the Clerk cast one ballot for the election of officers as named by the Nominating Committee.

Experiences during the recent Ohio River flood were related by E. Sherman Chase of Metcalf & Eddy, consulting engineers, and Dr. Gail P. Edwards and Arthur V. Harrington of the Massachusetts Department of Public Health. Information prepared by John F. McVey, Jr., of the Department of Public Health was presented by the Clerk of the section. After a short discussion, the meeting adjourned at 9.15 P.M.

RALPH M. SOULE, *Clerk*.

Designers Section

FEBRUARY 10, 1937. — A meeting of the Designers Section was held at the Society rooms this evening and was called to order at 6.35 P.M.

The minutes of the January meeting were read and approved.

The Vice-Chairman, Mr. Herman A. Dresser, called the attention of the members to the matter of increasing membership in the Society, and also in this Section, and suggested that the members obtain and circulate membership application blanks of which the Society has a supply.

The Chairman reminded the members of the joint meeting of the B. S. C. E. and the Northeastern Section, A. S. C. E., Student Chapters, to be held February 19, 1937, and also announced the date of March 10, 1937, for the next Section meeting, which will consider the use of aluminum in structural design.

The Chairman then introduced the speaker, Prof. Dean Peabody, Massachusetts Institute of Technology, who spoke on the subject, "The Effect of the American Concrete Institute's Reinforced Concrete Column Tests on Design." Professor Peabody described the comprehensive series of tests which have been made by the American Concrete Institute at the University of Illinois and at Lehigh University, and discussed the report of the committee, and its probable recommendations. This research has been confined to round columns with longitudinal steel, and tied with hoops and also with spirals.

The talk was illustrated with diagrams exhibited on the screen and a discussion period followed.

The meeting adjourned at 7.50 P.M., with an attendance of 40.

Respectfully submitted,
ANTHONY S. COOMBS, *Clerk*.

MARCH 10, 1937. — The annual meeting of the Designers Section was held at the Society rooms on March 10, 1937, at 6.40 P.M.

The report of the February meeting was read and approved, and the annual report of the Executive Committee for the past year was also read and approved.

The Chairman then called for the report of the Nominating Committee, which was presented, and which is filed with the records.

The Chairman called for further nominations from the floor, and there being none, it was voted that the Clerk cast one ballot for the nominees as presented by the Nominating Committee.

The Clerk cast one ballot for these nominees, as follows, and they were declared unanimously elected:

Chairman — Herman Dresser.
Vice-Chairman — Anthony Coombs.
Clerk — Donald T. Mitsch.
Executive Committee — Kimball Garland, A. E. Kleinert, Robinson Abbott.

The Chairman then introduced the speaker, Mr. B. J. Fletcher, Engineer in the Development Division of the Aluminum Company of America, New Kensington, Pennsylvania, who spoke on the subject "Recent Examples of the Design of Aluminum Alloy Structures." The talk was illustrated and treated of the uses and advantages of aluminum alloy structures, where weight saving is the controlling factor, and dealt particularly with excavating machinery booms and dippers, and also of light-weight bridge floors. Another important field for this alloy is for movable dams, where lightness and ease of handling are of more importance than first cost.

Mr. Fletcher answered numerous questions at the close of his talk, and at the conclusion of the meeting was tendered a rising vote of thanks.

The meeting adjourned at 8.15 P.M. with an attendance of 35.

Respectfully submitted,
ANTHONY S. COOMBS, *Clerk.*

Highway Section

JANUARY 27, 1937. — A meeting of the Highway Section of the Boston Society of Civil Engineers was held in conjunction with a meeting of the entire membership of the Boston Society of Civil Engineers on January 27, 1937, in the North Hall of Walker Memorial Building of the Massachusetts Institute of Technology. The meeting was called to order at 7.15 P.M. by the President of the Boston Society of Civil Engineers, Prof. Harold K. Barrows, who turned the meeting over to Prof. Henry B. Alvord, Chairman of the Highway Section, who introduced the speaker of the evening Mr. Charles R. Noble of the Port of New York Authority, who presented a paper on the subject, "The Design of Modern Express Highways from the Standpoint of Safety."

The paper was discussed by Professors Charles B. Breed and A. J. Bone and Messrs. Copell and Halsey.

A rising vote of thanks was tendered the speaker.

The meeting was adjourned at 9.50 P.M.

The attendance was 75.

Respectfully submitted,
T. C. COLEMAN, *Clerk.*

FEBRUARY 24, 1937. — A meeting of the Highway Section of the Boston Society of Civil Engineers was held in the Society rooms, 715 Tremont Temple, on Wednesday, February 24, 1937. The meeting was called to order at 7.30 P.M. by the Chairman, Prof. Henry B. Alvord.

The first business considered was the matter of election of officers for the coming year. A motion was made and passed that the Chairman appoint a Nominating Committee of three to present names of candidates for office for the coming year. The Chairman appointed Albert E. Kleinert, George A. Graves and John B. Babcock as a Nominating Committee.

The minutes of the previous meeting were read and accepted with slight changes.

The report of the Nominating Committee was then presented, offering the following candidates:

Chairman — A. J. Bone.
Vice-Chairman — T. C. Coleman.
Clerk — Thomas Giblin.
Executive Committee — Henry B. Alvord, George A. Graves, W. P. Morse.

A motion was made and carried that the nominations be closed. A motion was made and carried that the clerk cast one ballot for the officers as submitted by the Nominating Committee. This was done and the candidates named were declared elected.

The Chairman introduced the speaker of the evening, Prof. Charles B. Breed of the Massachusetts Institute of Technology, who presented a paper on the subject, "Causes and Control of Defects in Cement Concrete Pavements."

The speaker, displaying slides to illustrate his points, called the attention of his audience to the wide variation in the results of tests on different samples of concrete taken under similar conditions, and emphasized the necessity of uniformity of product. He mentioned the difficulty of securing uniformity of aggregate, and recommended that resident engineers be equipped with the necessary authority to correct for varying aggregate before work has progressed far enough to cause damage.

The speaker showed the correct and incorrect use of machinery to place the concrete, and spoke of the precautions to be taken to avoid the separation of the aggregate during the process. He also demonstrated the methods of finishing the surface and the joints in order to avoid the defects that are more or less common at these points.

He also spoke of the use of calcium chloride and its effect on the surface. Slides were displayed showing various kinds and degrees of failure of concrete pavements produced by the different causes.

Following the presentation of the paper there was an interesting discussion of the points raised.

The speaker was accorded a rising vote of thanks. The meeting adjourned at 9.30 P.M.

Respectfully submitted,
T. C. COLEMAN, *Clerk.*

Northeastern University Section

FEBRUARY 25, 1937. — The Northeastern University Section of the Boston Society of Civil Engineers held a regular meeting in Room 18–H, Huntington Building. Twenty-seven members and guests were present, and 11 attended the dinner at Child's Old France Restaurant.

The meeting was called to order at 7.30 P.M. by the Chairman, G. Luther Cheney, and the minutes of the October meeting were read and approved. A Nominating Committee to act upon Division A officers for the coming year was appointed, as follows: Lawrence M. Peterson, Alfonso M. Cavaliere and Harold E. Cobb.

Mr. J. Stuart Crandall, President of the Crandall Dry Dock Engineers, spoke on "Pile Foundations." Various load conditions, with the effect on the several types of piles, together with the resulting earth effects, were discussed by the speaker. A very interesting portion of the talk consisted in the detailed explanation of the two existing pile formulæ, namely, the Engineering News Record and the Hiley formulas.

After a short group discussion, following the speaker's address, the meeting adjourned at 9.30 P.M.

ROBERT WM. SMITH, *Clerk.*

MARCH 11, 1937. — The Northeastern University Section of the Boston Society of Civil Engineers held a special business meeting in Room 201–L, Laboratory Building, and 28 were present.

The meeting was called to order at 12.05 P.M. by a member, A. M. Cavaliere. At this time an election of officers was held. Those men elected for the coming year in Division A are as follows:

Chairman — Chesley F. Garland.
Clerk — Harold E. Sanford.
Executive Committee — Charles H. Fish and Arthur E. Gibson.

The meeting adjourned at 12.45 P.M., after a short discussion.

ROBERT WM. SMITH, *Clerk.*

APPLICATIONS FOR MEMBERSHIP

[May 1, 1937]

THE By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission.

Anderson, Paul N., Worcester, Mass. (Age 28, b. Worcester, Mass.) In 1929–30, detailer and supervisor with G. Adolph Johnson, architect, Worcester; 1930–31, chief construction engineer, Wahl Construction Company, Chicago, on new Worcester Post Office and Court House; 1931–32, estimator and supervisor with August Scott, contractor, Worcester; also chief layout engineer and assistant to chief engineer on Worcester Municipal Memorial Auditorium, George A. Fuller Construction Company; 1932–33, 1933–34, 1934–35, city of Worcester engineering department, (1) as chief of party in field on block survey of city; (2) chief of party, director of squad and inspector of street construction; (3) senior engineer, in charge of all field work mapping, zoning and studying street conditions; 1935–36, observer on traverse and triangulation survey in Worcester County; 1936–37, senior engineer on the Blackstone Valley rehabilitation project, town mapping and survey of mills, also in charge of field parties on projects for flood control in the watershed area of the Blackstone River, for the State Planning Board, Boston. Refers to *C. M. Allen, A. W. French, J. W. Howe, R. A. Marble, E. A. Taylor.*

Benford, William R., North Providence, R. I. (Age 35, b. Providence, R. I.) Graduate of Brown University, 1927, S.B. in engineering; 1927–35, instructor in civil engineering, Brown University; also during same period, associated with Leighton T. Bohl on general practice, including surveying, plot development, design and supervision of structures, testing of materials, and employed by various architects as draftsman and superintendent of construction; 1935 to date, assistant professor of civil engineering, Brown University. Refers to *A. Casagrande, G. M. Fair, A. Haertlein, F. E. Winsor.*

Dietz, Albert George, Boston, Mass. (Age 29, b. Lorain, Ohio.) Liberal arts course, Miami University, Oxford, Ohio, 1926–29, mathematics and German; 1929–32, at Massachusetts Institute of Technology, degree of B.S. in building construction; 1932, Massachusetts Institute of Technology; also in 1930, A.B. degree awarded by Miami University; 1936, degree of M.S. at Massachusetts Institute of Technology; 1932–33, construction work and in steel mills, National Tube Company; 1934–36, assistant, and 1936 to date, instructor in building construction, department of civil engineering, Massachusetts Institute of Technology. Refers to *C. B. Breed, D. Peabody, K. C. Reynolds, W. C. Voss.*

Edwards, Lillian F., South Orleans, Mass. (Age 28, b. Orleans, Mass.) Graduated from Orleans High School in 1927 with honors; special tutoring in higher mathematics. About ten years of theoretical and practical experience in office of Arthur L. Sparrow, civil engineer, Orleans, Mass., making computations, plans and tracings; also as head of survey party in making Land Court and topographical surveys, subdivisions, highway layouts, etc. At present an assistant, employed by Arthur L. Sparrow. Refers to *N. Bearse, W. C. Hawley, C. B. Humphrey, A. L. Sparrow.*

Enebuske, Viking, Boston, Mass. (Age 46, b. Cambridge, Mass.) Graduated, 1915, from Massachusetts Institute of Technology; S.B. degree in civil engineering; July, 1915, to April, 1916, junior assistant engineer, Public Service Commission of New York, on field work on East River tunnels: April, 1916, to October, 1917, draftsman and assistant engineer, Pittsburgh Des Moines Steel Company, Pittsburgh, Pa.; October, 1917, to August, 1919, United States Army war service; August, 1919, to October, 1923, assistant engineer, Foundation Company of New York, at the Pittsburgh district office, on structural design and estimates of cost; January, 1924, to August, 1924, concrete designer, Dwight P. Robinson Company, New York City; August, 1924, to October, 1925, assistant estimator, White Construction Company, New York City; October, 1925, to September, 1931, assistant engineer, Guggenheim Brothers, New York City, in charge of estimates on nitrate development projects in Chile, South America; February to March, 1932, inspections and engineering reports for department of industrial research, Massachusetts Institute of Technology, covering municipal buildings in Cambridge, Mass.; November to December, 1933, assistant engineer, Guggenheim Brothers, New York City; October to December, 1936, structural designer with Le Roy M. Hersum, consulting engineer, Boston; December, 1936, to date, structural designer, Stone & Webster Engineering Corporation, Boston, Mass. Refers to *H. G. Dresser, L. M. Hersum, E. A. Norwood, J. R. Worcester.*

HOOPER, LESLIE JAMES, Holden, Mass. (Age 34, b. Essex, Mass.) Education, public schools, Millbury, Mass.; graduated from Worcester Polytechnic Institute, degree of B.S. in mechanical engineering in 1924, and degree of mechanical engineer in 1928. During three years following graduation in employ of Canadian and General Finance Company, Ltd., as hydraulic testing engineer on various Brazilian properties, including field tests of waterwheels, penstocks and governors. Since 1927 employed by Prof. C. M. Allen on field and laboratory work, including principally salt velocity tests and hydrometer tests of waterwheels, calibration of hydraulic instruments and model tests. Since 1932 instructor at Worcester Polytechnic Institute. From 1934 to date, John R. Freeman Scholar, Boston Society of Civil Engineers. Author of various papers on hydraulic subjects. Refers to *C. M. Allen. C. R. Bliss, A. W. French, J. W. Howe, C. T. Main.*

LEGRO, HAROLD WILFRED, Peabody, Mass. (Age 49, b. Lynn, Mass.) Graduated from Lynn English High School, 1907. Employed as rodman, engineering department, Boston & Maine Railroad. beginning 1909; served as transitman and instrumentman until 1918, on various railroad projects, including double track and engine terminal construction, ballasting programs, and other maintenance engineering; 1918–21, assistant track supervisor on maintenance of way work; 1921–28, assistant division engineer in charge of engineering design in part and layout of divisional maintenance of way projects; until the present, industrial engineer, Boston & Maine Railroad, handling engineering matters in connection with industrial developments; member of American Railway Engineering Association since 1926, and Chairman of subcommittee on properties of earth materials of the Roadway Committee, since 1932. Refers to *W. J. Backes, W. F. Cummings, B. W. Guppy, A. Haertlein.*

MACDONALD, WALTER F., Wollaston, Mass. (Age 50, b. East Boston, Mass.) Education, Boston public schools, Mechanic Arts High School, Franklin Union and Boston evening schools. Employed twenty-nine years by Chas. T. Main, Inc., as draftsman, inspector and resident engineer on mills, dams, hydro-electric stations and steam power plants; at present in general contracting business, under name of Vappi & Macdonald Company, Inc., Cambridge, Mass. Refers to *F. M. Gunby, C. R. Main, C. T. Main, H. E. Sawtell, W. F. Uhl.*

MARBLE, ALDEN SHAILER, Fitchburg, Mass. (Age 30, b. Estancia, New Mexico.) Education, two and a half years (non-technical), at Antioch College, Ohio, leaving in 1924, followed by individual study of technical subjects; 1926–30, on construction work with Western Union Telegraph Company, headquarters at Cleveland and Chicago; then three years' general engineering experience with Parker, Bateman & Chase (now R. A. Marble), Fitchburg, Mass., including duties as transitman, chief of party, drafting computations and design (1931–33, inclusive); October, 1933, to March, 1934, as chief of party on layout of streets, buildings and miscellaneous construction at Fort Devens, Mass., under Construction Quartermaster; April, 1934, to November, 1934, with Parker, Bateman & Chase; December, 1934, to December, 1935, with Fitchburg City Planning Board, on design, layout and inspection; January, 1936, to date, with engineering division, Department of Public Works, Fitchburg, Mass., transitman, conducting field party, designer of bridges and other construction work connected with city engineering office. Refers to *H. B. Allen, E. Jacoby, R. A. Marble, C. E. Pethybridge.*

Parker, Franklin Peabody, Wellesley Hills, Mass. (Age 23, b. Salt Lake City, Utah.) Graduated from Massachusetts Institute of Technology, degree of B.S. in architectural engineering, 1936. At present with Chas. T. Main., Inc. Refers to *F. M. Gunby, L. G. Ropes, T. H. Safford, W. F. Uhl.*

Swartz, Hermon S., Newton, Mass. (Age 32, b. Newton, Mass.) Graduated from Newton High School, 1923, and from Massachusetts Institute of Technology in 1929; 1923–29, part time, T. Stuart & Son Company; 1929–34, engineer with Patrick McGovern, Inc., New York City; 1935, with American Oil Company; 1936–37, Editor of "New England Construction." Refers to *J. B. Babcock, M. N. Clair, C. A. Farwell, C. M. Spofford.*

Wiggin, David Charles, Jr., Cambridge, Mass. (Age 23, b. Peabody, Mass.) Graduated from Tufts College, degree of B.S. in civil engineering, in 1935; and M.S. in sanitary engineering at Harvard in 1936. At present, assistant in sanitary engineering department, Harvard University, Cambridge, Mass. Refers to *H. P. Burden, G. M. Fair, A. Haertlein, F. N. Weaver.*

For Transfer from Grade of Junior

Frost, Daniel Carter, Bloomfield, N. J. (Age 33, b. Holliston, Mass.) Graduated from Northeastern University, degree of B.C.E. in 1926; degree of C.E., 1928, Thayer School of Civil Engineering, Dartmouth College; 1926–27, Massachusetts Department of Public Works, inspector and transitman; May, 1928, to September, 1928, chief of party, Vermont State Highway Department on relocation of flooded main highways; winter of 1928–29, teacher of mathematics, Burlington, Vt., high school; summer of 1929, Pittsburgh Des Moines Steel Company, Pittsburgh, draftsman and detailer; September, 1929, to February, 1931, instructor in surveying and mathematics at Beloit College, Beloit, Wis.; February, 1931, to June, 1936, at Patterson East Side High School, teaching in department of mathematics and drafting; September, 1936, to date, at Newark College of Engineering, Newark, N. J., instructor in civil engineering; also from September, 1931, to date, teacher in mechanics and mathematics, Newark Technical School, evening school, Newark, N. J. Refers to *H. B. Alvord, C. S. Ell, F. F. Hampe, J. M. Robbins.*

Maccario, Charles Victor, Malden, Mass. (Age 28, b. Boston, Mass.) Graduated from Malden High School, 1927; graduated from Northeastern University, degree B.C.E., in 1931, and B.S. in 1932. While at Northeastern University, co-operative periods, was transitman and draftsman, with H. Richardson, highway surveyor, Harwich and Brewster, Mass.; transitman and inspector, town of Swampscott, Mass.; transitman and chief of party, Department of Public Works, Melrose, Mass.; 1933–34, assistant engineer, city engineering department, Malden, Mass.; 1934 to date, estimator and supervisor of construction with G. Maccario & Sons, civil engineers and contractors, Malden, Mass. Refers to *H. B. Alvord, C. O. Baird, Jr., C. S. Ell, A. E. Everett, Jr., E. J. O'Connor.*

Verner, Robert Henry, Memphis, Tenn. (Age 30, b. Erving, Mass.) Graduated from Northeastern University, degree of B.C.E. in 1931. During co-operative periods at Northeastern University, 1928–31, rodman, transitman and draftsman; 1931 to date, with United States Engineers at Memphis, Tenn., as inspector on levee construction and dredging operations, and resident engineer on channel improvement project; at present, in office, planning and estimating dredging work. Refers to *H. B. Alvord, C. O. Baird, Jr., A. E. Everett, Jr., E. N. Hutchins.*

Transfer from Grade of Student

Gale, Howard E., Jr., Dorchester, Mass. (Age 29, b. Somerville, Mass.) Graduated from Northeastern University, 1936. From 1926 to 1931, drafting, with Hunt-Spiller Manufacturing Company; 1935–36, drafting and transitman, New England Survey Service; 1936 to date, junior engineering aid, Metropolitan District Water Supply Commission. Refers to *H. B. Alvord, C. O. Baird, Jr., L. M. Clark, A. E. Everett, Jr.*

Redfield, William A., Middletown, N. Y. (Age 27, b. Bloomingburg, N. Y.) Graduated from Northeastern University, degree of B.S. in civil engineering, 1936; surveying at various times, total of five years, with Harry C. Carpenter, civil engineer, Middletown, N. Y.; autumn of 1932, on boundary survey, city of Middletown, N. Y.; six months, summer of 1934, on construction and maintenance department, Colonial Beacon Oil Company, Boston, Mass.; part of winter and all summer, 1935, under H. Eltinge Breed, in Highway Department, Orange County, New York; during junior year at Northeastern, junior field instructor; July to November, 1936, with North-Eastern Construction Company, New York City, at Roanoke, Va., on construction of Veterans Administration Facility building, as field engineer, estimator, inspector and draftsman; at present with White Construction Company, New York City, at Northport, N. Y., as field engineer on construction of Veterans Administration Facility buildings. Refers to *H. B. Alvord, C. O. Baird, Jr., C. S. Ell, A. E. Everett, Jr.*

Zacher, Robert C., Irvington, N. J. (Age 35, b. Irvington, N. J.) Graduated from Merrill High School, Irvington, N. J., in 1932; graduated from Northeastern University in 1935. During summers of 1931–35, employed by Zacher & Kurrle, Inc., builders, Newark, N. J.; May, 1935, to May, 1936, employed as estimator, detail drawing and general supervision; May, 1936, to date, with J. R. Steers, Inc., contracting engineers, New York City, or Newark, N. J., Pennsylvania Railroad improvements. Refers to *H. B. Alvord, C. O. Baird, Jr., C. S. Ell, A. E. Everett, Jr.*

ADDITIONS

Members

Milton J. Adams, 65 Edwin Street, Atlantic, Mass.
John S. Brinkler, 15 Goldthwait Street, Lynn, Mass.
John P. Cronin, 79 Hillsdale Road, Medford, Mass.
Philip Dixon, 57 Intervale Street, Roxbury, Mass.
John L. Elliott, 404 Waltham Street, West Newton, Mass.
Emil A. Gramstorff, 19 Hilltop Avenue, Lexington, Mass.
Carl A. Held, 519 Church Street, Bound Brook, N. J.
Richard W. Johnson, 203 High Street, Newburyport, Mass.
Francis M. Mahard, 10 Parkman Street, Natick, Mass.
William T. Morrissey, 1103 Morton Street, Dorchester, Mass.
Bernard J. Parker, 334 Massachusetts Avenue, Boston, Mass.
Herman G. Protze, 76 Farquhar Street, Roslindale, Mass.
Harald M. Westergaard, 33 Pinehurst Road, Belmont, Mass.

Associate Member

Stuart Huckins, 33 Bancroft Road, Wellesley Hills, Mass.

Junior

John B. Ledden, 74 Elizabeth Street, Fitchburg, Mass.

Students

Arnold B. Davis, 35 Pulsifer Street, Newtonville, Mass.
John C. Finn, 22 Dunreath Street, Roxbury, Mass.
Charles H. Fish, 38 Chester Avenue, Waltham, Mass.
Chesley F. Garland, 12 Lewis Road, Belmont, Mass.
Joseph Higgins, 6 Marshall Street, Revere, Mass.
John M. Squarebriggs, 55 Vernon Street, Brookline, Mass.
Peter R. Vandersloot, 30 Cherry Place, West Newton, Mass.

DEATHS

George H. Brazer . . April 20, 1937
John H. Gregory . . Jan. 18, 1937

BOOK REVIEW*

"*Theory of Continuous Structures and Arches, March, 1937, by Charles M. Spofford. McGraw-Hill Book Company, New York. 267 pages. Price, $3.50.*

The development and simplification of methods of stress analysis for statically indeterminate structures in recent years is one of the factors which has tended to overcome prejudice against the use of continuous truss bridges in the United States. The author has been an advocate of continuous bridges, and his faith in such structures is manifested by the number of large bridges of this type which have been built under his direction in the last few years. The book, therefore, has a solid practical basis.

The book opens with a presentation of several methods for determining reactions in continuous beams or girders. There follows a detailed discussion of continuous truss analysis. Basic equations are developed and put in such form that the ensuing numerical work can be performed easily in a suggested tabulated form. Preparing influence tables for a continuous truss with about one hundred and fifty members ordinarily might seem a staggering undertaking. The recommended tabulations tend to avoid confusion and to reduce the labor.

Effect of pier settlements and also cambering and erection adjustments are discussed. There is also a brief presentation of statically determinate continuous trusses of the Wichert and Rhomboid types.

The theory of arches, which comprises about 75 per cent of the book, is introduced by a chapter on the distribution of normal stress on the cross section of straight and curved bars. In the case of straight bars a general theory of flexure is developed which is applicable to bars of non-symmetrical section.

The historical sketch includes an interesting illustrated reference to recently excavated brick arches which were built more than five thousand years ago.

The arch theory deals chiefly with two-hinged and hingeless arches of the rib or solid web type. For the two-hinged parabolic arch, formulas are developed for the solution of specific problems, such as the location of the section of maximum bending moment, the determination of the maximum stress at the quarter-point, and determination of the average axial thrust.

In dealing with two-hinged segmental arches a precise method is used in which there is included the effect of curvature of the arch axis on the stress distribution over the cross section. Application of the resulting formulas is simplified by use of a table. The circular arch is not generally used in practice. However, the mathematical development shows what can be accomplished in the way of precise analysis, and affords a means of comparison with the results of approximate methods.

Deflection of two-hinged and three-hinged arches is treated in a manner probably not found elsewhere in English literature.

The part on fixed-ended arches includes a new treatment of continuous arches on elastic piers based on the method of least work. The solution involves equations which are used in a manner similar to that in which the three-moment equations are used for continuous straight beams.

A chapter on analysis of rigid frame bridges is included.

Application of the theory is well illustrated by numerical examples of a practical nature. Each chapter ends with a number of problems and a bibliography. There are several illustrations, taken largely from structures designed by the author and his associates. The book is well arranged as a text, with excellent typography and paper, and should be useful also for reference.

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* By Eugene Mirabelli, Assistant Professor of Structural Design, Massachusetts Institute of Technology, Cambridge, Mass.

ANNUAL REPORTS

Report of Board of Government for the Year 1936–37

BOSTON, MASS., March 17, 1937.

To the Boston Society of Civil Engineers:

Pursuant to the requirements of the By-Laws, the Board of Government presents its report for the year ending March 17, 1937.

Membership

Eleven new members, 4 juniors, 1 associate and 23 students have been added during the year, and 6 members and 5 juniors have been reinstated, making a total addition of 50 members.

During the year 14 members have died, 13 have resigned, 4 members have had dues remitted and resignations accepted, 58 members have been dropped for non-payment of dues, making a total deduction of 89.

The present net membership of the Society consists of 5 honorary members, 601 members, 75 juniors, 62 students, 9 associates, 1 member of Sanitary Section, making a total membership of 753, a net loss for the year of 39.

The honorary membership list now is as follows:

Charles T. Main, elected January 28, 1932.
Dr. Karl T. Compton, elected February 17, 1932.
Prof. C. Frank Allen, elected March 16, 1932.
Harrison P. Eddy, elected February 21, 1934.
Joseph R. Worcester, elected February 21, 1934.

Deaths

Members:
- Charles H. Bartlett, March 1, 1937.
- Philip D. Borden, December 24, 1936.
- John C. Chase, April 15, 1936.
- Frank P. Cobb, September 23, 1936.
- Edmund S. Davis, December 17, 1936.
- Luther Dean, January 9, 1937.
- John H. Gregory, January 18, 1937.
- Frank S. Hart, July 11, 1936.
- Lewis M. Hastings, April 25, 1936.
- Edward H. Kitfield, June 26, 1936.
- Leon O. Norwood, June 26, 1936.
- Frank B. Rowell, May 31, 1936.
- Frank O. Whitney, May 13, 1936.
- James W. Woodhill, October 5, 1936.

Remission of Dues and Extension of Time

The membership of the Society has been subject during recent years to the effects of the depression. During this period the Board of Government has granted a number of members an extension of time for payment of dues and has remitted the dues of other members. With much regret the Board voted this year to drop from membership those who had not paid their 1936 dues for which an extension had been granted. The Board has remitted dues and accepted resignations from 4 members. In the case of 3 members, dues for the year ending March 17, 1937, have been remitted.

The Board has granted 30 members an extension of time for payment of dues for year ending March 17, 1937.

Exemption of Dues

Eighty-two members are now exempt from dues in accordance with By-Law 8, which provides that "a member of any grade who has paid dues for forty years, or who has reached the age of seventy years and has paid dues for thirty years, shall be exempt from further dues."

Meetings of the Society

Eight regular meetings, since the annual meeting, have been held during the year. An excursion, held on June 26, 1936, attended jointly by members of the Society and by members of the International Conference on Soil Mechanics and Foundation Engineering, was made to the Ware and Swift River projects of the Metropolitan District Water Supply Commission.

The November meeting was the Annual Student Night, attended by Student Chapters, A. S. C. E., at Harvard, Massachusetts Institute of Technology, Tufts, Brown and Rhode Island State College, and the Northeastern University Section, B. S. C. E.

The February meeting was a joint meeting with the Northeastern Section of the American Society of Civil Engineers and the Student Chapters, A. S. C. E., at Harvard, Massachusetts Institute of Technology and Tufts, and the Northeastern University Section, B. S. C. E.

The total attendance at the eight meetings was 1,717 persons; the largest attendance was 950 and the smallest 50. Buffet suppers have been a feature of seven meetings and they were well attended. The papers and addresses given were as follows:

March 18, 1936. — Annual meeting. Address of retiring President, John B. Babcock, 3d, "The Boston Society of Civil Engineers and its Founder Members," followed by dinner, smoker and entertainment.

April 15, 1936. — "The March, 1936, Flood," a symposium by H. B. Kinnison, District Engineer, Water Resources Branch, United States Geological Survey, Boston; Arthur T. Safford, Chief Engineer, and S. Stanley Kent, Assistant Engineer, both of Proprietors of Locks and Canals on Merrimack River, Lowell; Richard S. Holmgren, Chief Engineer, New Hampshire Water Resources Board, Concord, N. H.; Harry N. Nelson, Hydraulic Engineer, New England Power Engineering and Service Corporation, Boston; A. W. Ladd, Hydraulic Engineer, Holyoke Water Power Company, Holyoke, Mass.

May 20, 1936. — "A Fundamental Fallacy in Earth Pressure Computations," by Dr. Karl Terzaghi.

June 26, 1936. — Joint Excursion of Boston Society of Civil Engineers and Members of the International Conference on Soil Mechanics and Foundation Engineering (held at Harvard University) to the Ware and Swift River projects of the Metropolitan District Water Supply Commission.

September 23, 1936. — "North Metropolitan Sewerage Problems," by Joseph P. Dever, Chief Engineer and Director, Sewer Division, Metropolitan District Commission.

October 21, 1936. — "Rouses Point Bridge across Lake Champlain, between Rouses Point, New York, and Arlburg, Vermont;" general description of project by Carroll A. Farwell; design of the bridge structure by H. J. Williams.

November 19, 1936. — "Exploratory Surveys along the Alaska-Yukon Boundary," by Bradford Washburn, Instructor in Geographical Explorations at Harvard University (Student Night).

December 16, 1936. — "American Hydraulic Laboratory Practice," by Leslie J. Hooper, Instructor in Hydraulic Engineering at the Alden Hydraulic Laboratory of Worcester Polytechnic Institute, Worcester, Mass.

January 27, 1937. — "The Factor of Safety in Highway Design," by Charles M. Noble, of the Staff of the Port of New York Authority (joint with Highway Section and Student Chapters, A.S.C.E., and Northeastern University Section, B.S.C.E.).

February 19, 1937. — "The San Francisco–Oakland Bay Bridge," by Charles F. Goodrich, Chief Engineer of the American Bridge Company.

Sections

Nineteen meetings were held by the Sections of the Society during the year. These meetings of the Sections, offering opportunity for less formal discussion, have continued to demonstrate their value to their members and to the Society. The variety of subjects presented has made an appeal to the members, as indicated by the general attendance at these meetings.

Sanitary Section Meetings. — The Sanitary Section has held 4 meetings during the year, with an average attendance of 30. The papers and meetings are listed in the report of the executive committee. Present membership, 165.

Designers Section Meetings. — The Designers Section has held 8 regular meetings during the year. The average attendance has been 56. The papers and meetings are listed in the report of the executive committee. Present membership 129.

Highway Section Meetings. — The Highway Section has held 4 regular meetings during the year, with an average attendance of 46. Present membership 87.

Northeastern University Section. — The Northeastern University Section has held 3 meetings during the year, with an average attendance of 59. Present total membership 180, of which 57 are in attendance at the University.

Journal

The complete report of the Editor of the Journal for the calendar year 1936 will be printed in the April, 1937, Journal.

Funds of the Society *

Permanent Fund. — The Permanent Fund of the Society has a present value of $50,000. The Society again authorized the use of as much as necessary of the current income of this fund in payment of current expenses.

John R. Freeman Fund. — In 1925 the late John R. Freeman, a Past President and honorary member of the Society, made a gift to the Society of securities which was established as the John R. Freeman Fund, the income from which was about $1,300. The income from this fund is to be particularly devoted to the encouragement of young engineers. Mr. Freeman suggested several uses, such as the payment of expenses for experiments and compilations to be reported before the Society; for underwriting meritorious books or publications pertaining to hydraulic science or art; or a portion to be devoted to a yearly prize for the most useful paper relating to hydraulics contributed to this Society; or establishing a traveling scholarship every third year open to members of the Society for visiting engineering works, reports of which would be presented to the Society.

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* Details regarding the values and income of these funds are given in the Treasurer's report.

Edmund K. Turner Fund. — In 1916 the Society received 1,105 books from the library of the late Edmund K. Turner, and a bequest of $1,000, "the income of which is to be used for library purposes." The Board voted to use $50 of the income for the purchase of books for the library.

Alexis H. French Fund. — The Alexis H. French Fund, a bequest amounting to $1,000, was received in 1931 from the late Alexis H. French of Brookline, a former Past President of the Society. The income of this fund is "to be devoted to the library of the Society." The Board voted to use a portion of the available income for permanent binding of certain periodicals.

Desmond FitzGerald Fund. — The Desmond FitzGerald Fund, established as a bequest from the late Desmond FitzGerald, a Past President and honorary member of the Society, provides that the income from this fund shall "be used for charitable and educational purposes." The Board voted on January 22, 1936, to appropriate from the income of this fund the sum of $100, to be known as the Boston Society of Civil Engineers Scholarship in memory of Desmond FitzGerald, and to be given to a student of Northeastern University. Presentation of this scholarship was made by President H. K. Barrows at a student meeting of the University on April 22, 1936, to James L. Dallas of Beverly, Massachusetts, a senior student in civil engineering. This year, on January 27, 1937, the Board voted to award a scholarship which will be presented to the selected student at a special meeting of the students to be held at an early date at the University.

Tinkham Memorial Fund. — The "Samuel E. Tinkham Fund," established in 1921 at the Massachusetts Institute of Technology by the Society "to assist some worthy student of high standing to continue his studies in civil engineering," had a value of $2,477.64 on June 30, 1936. Mr. Thomas Evans of Quincy, Massachusetts, a student in civil engineering in the class of 1938 at Technology, has been awarded this scholarship for the year 1936–37.

Clemens Herschel Fund. — This fund was established in 1931 by a bequest from the late Clemens Herschel, a former Past President and honorary member of the Society. The income from this fund is to be used for the presentation of prizes for particularly useful and commendable papers presented at meetings of the Society. The present value of this fund is about $1,000. The expenditure made during the year from this fund was for prizes.

Edward W. Howe Fund. — This fund, a bequest of $1,000, was received December 2, 1933, from the late Edward W. Howe, a former Past President of the Society. No restrictions were placed upon the use of this money, but the recommendation of the Board of Government is that the fund be kept intact, and that the income be used for the benefit of the Society or its members. The expenditure made during the year was very appropriately for the preparation of photographs and cuts for the publication of the Address at the Annual Meeting, March 18, 1936, of President John B. Babcock, on "The Boston Society of Civil Engineers and its Founder Members," published in the July, 1936, JOURNAL.

Special Prize

A paper of particular value to the membership at large was selected this year by the Board of Government, upon recommendation of the Committee on Prize Awards, for a special prize, namely, the Presidential Address of Past President John B. Babcock, 3d, on "The Boston Society of Civil Engineers and its Founder Members," delivered at the Annual Meeting on March 18, 1936, and published in the July, 1936, JOURNAL. The prize consisted of books, as follows: "Fifty Years of Boston," memorial volume issued in commemoration of the Tercentenary of 1930; "Elements of Railroad Engineering," by W. G. Raymond, revised by Riggs and Sadler; "Early History of Modern Civil Engineering," by Kirby and Lawson.

Desmond FitzGerald Medal

The Desmond FitzGerald Medal (bronze) was provided for in 1910 as an endowed prize by the late Desmond FitzGerald, a former Past President and honorary member of the Society. The prize is awarded annually to a member who presents an original paper to the Society which is published in the JOURNAL for the current year.

In accordance with the recommendation of the Committee on Awards, the Board of Government voted to award the Desmond FitzGerald medal to Albert Haertlein, member, for his paper on "The Design of Statically Indeterminate Trusses," presented at a meeting of the Designers Section, held on January 8, 1936, and published in the JOURNAL for April, 1936.

Section Prizes

The Board of Government voted on April 12, 1924, to present a prize for a worthy paper given in each section by a member of that section, "this award to consist of books suitably inscribed."

Sanitary Section Prize. — The Board adopted the recommendation of the Sanitary Section Prize Committee, and voted that the Sanitary Section Prize be awarded to Almon L. Fales, member, for his paper on "Chemical Precipitation of Sewage and Industrial Wastes," presented at a meeting of the Sanitary Section on April 1, 1936, and published in the October, 1936, JOURNAL. The prize consisted of Merriman's "Civil Engineers Handbook" and "Handbook of Chemistry and Physics," edited by Charles D. Hodgman, M.S.

Designers Section Prize. — The Board adopted the recommendations of the Designers Section Prize Committee, and voted that the Designers Section Prize be awarded to Herman G. Dresser, member, for his paper on "Some Features of Sewer and Culvert Design," presented at a meeting of the Designers Section on March 11, 1936, and published in the October, 1936, JOURNAL. The prize consisted of the following books: "Rahmenformeln," by Kleinlogel; "Continuous Frames of Reinforced Concrete," by Cross and Morgan; and "Vibration Prevention in Engineering," by Arthur L. Kimball.

Clemens Herschel Award

The late Clemens Herschel, a former Past President and honorary member, gave to the Society a number of copies of his book on "Frontinus and the Water Supply of the City of Rome," with the request that the Board award one or more of the books each year as prizes for papers which have been particularly useful and commendable and worthy of grateful acknowledgment.

On recommendation of the Desmond FitzGerald Committee on Awards, the Board made no award this year.

Social Activities

Five of the regular meetings have been held at the Engineers' Club and one meeting at Chipman Hall, Tremont Temple, preceded by buffet suppers, under the direction of the Social Activities Committee. As usual, the most enthusiastic meeting was Student Night, held in November, and attended by members of the Student Engineering Societies of Massachusetts Institute of Technology, Harvard, Tufts, Rhode Island State College, Brown University and Northeastern University Section of the Boston Society of Civil Engineers. An excursion was made to the Metropolitan District Water Supply Commission projects on the Ware and Swift Rivers on June 26, 1936; 125 persons attended.

Library

The Report of the Committee on Library contains a complete account of the library activities during the past year.

Society Activities

The usual special committees dealing with the general activities and conduct of the Society have included the following: Program, Publication, Membership and Publicity, Library, Social Activities, Relation of Sections to Main Society, and Membership Records. Other special committees have included the following: Run-off, 1936 Floods, Desmond FitzGerald and other prizes, and John R. Freeman Fund and Welfare Committee. Each of these committees has made a distinct contribution to the Society and has developed fields of endeavor which will prove of great benefit to the members.

The Society has co-operated with the Engineering Societies of New England, and many members of the Society have served on Engineering Societies' committees.

Your Board, in conclusion, wishes to express its appreciation of the excellent work done by the officers of the Sections and by the committees of the Society.

H. K. BARROWS, *President.*

Report of the Treasurer

BOSTON, MASS., March 10, 1937.

To the Boston Society of Civil Engineers:

During the past year the financial records of the Society have been carried forward in the manner established by your former Treasurer, and are shown on the enclosed tables, as follows:

Table 1. Distribution of Funds — Receipts and Expenditures.
Table 2. Record of Investments.

It will be noted that it has been necessary to transfer $1,448.85 of the income of the Permanent Fund to the Current Fund to meet current expenses and to re-establish the Current Fund at $1,500. It is gratifying, however, to note at least a small increase in receipts of dues from members.

	1932-33	1933-34	1934-35	1935-36	1936-37
Net expenditures (except such as provided for by appropriations other than Permanent Fund income)	$8,734	$7,228	$7,609	$6,333	$6,807
Receipts from dues	6,030	5,540	5,390	5,115	5,358
Deficit	$2,704	$1,688	$2,219	$1,218	$1,449

Following the practice established last year, the incomes from all investments have been pooled and allocated to the various funds on the basis of respective book values at the beginning of the year. Interest and dividends for the year totaled $4,477.57, being 5.185 per cent of the total book value at the beginning of the year, a slightly higher return than for the previous year. Several changes in the schedule of investments, as recommended by the Investment Committee, have been made during the year. These are shown in detail in Table 2. The changes in the two principal funds during the past five years are given below:

	March 10, 1933	March 10, 1934	March 10, 1935	March 10, 1936	March 10, 1937
Permanent Fund:					
Book value	$50,263	$50,348	$51,932	$52,695	$54,865
Market value in per cent of book value	85.42%	91.42%	90.92%	99.87%	103.90%
John R. Freeman Fund:					
Book value	$26,200	$26,902	$26,999	$27,359	$29,073
Market value in per cent of book value	67.43%	66.01%	61.12%	99.87%	103.90%

The total book value of all holdings on March 10, 1937, is $91,866.40, not including the value of the library and furniture, which for many years has been carried on the books at $9,905.11.

During the past year it has been necessary for the Treasurer to be absent from Boston much of the time. While this has resulted in some inconvenience, we have been fortunate in having the help of Mr. Karl R. Kennison, who very kindly consented to serve as Acting Treasurer to take care of any emergency and to represent the treasurer in his absence.

Respectfully submitted,

CHARLES R. MAIN, *Treasurer.*

TABLE 1. — DISTRIBUTION OF FUNDS — RECEIPTS AND EXPENDITURES.

	Book Value March 10, 1936	Interest and Dividends	Net Profit (or Loss) at Sale or Maturity	TRANSFER OF FUNDS		Book Value March 10, 1937
				Purchased	Sold	
Bonds	$38,728 51	$1,771 14	$479 08	$6,341 25	$13,312 50	$32,236 34
Co-operative banks	11,901 80	494 24	–	7,391 00	5,000 00	14,292 80
Stocks	35,129 02	2,212 19	452 19	12,924 40	3,575 52	44,930 09
Cash (except Current Fund)	595 35	–	–	–1,688 18*	–	–1,092 83
Total (except Current Fund)	$86,354 68	$4,477 57	$931 27	$24,968 47	$21,888 02	$90,366 40

	Book Value March 10, 1936	ALLOCATION OF THE ABOVE —		Miscellaneous Receipts	Miscellaneous Expenditures	Book Value March 10, 1937
		At 5.185%	At 1.078%			
Permanent Fund	$52,694 82	$2,732 25	$568 23	$319 00†	$1,448 85	$54,865 45
John R. Freeman Fund	27,359 26	1,418 60	295 11	–	–	29,072 97
Edmund K. Turner Fund	1,000 00	51 85	10 78	–	47 95	1,014 68
Desmond Fitzgerald Fund	2,092 44	108 50	22 56	–	100 00	2,123 50
Alexis H. French Fund	1,000 00	51 85	10 78	–	35 00	1,027 63
Clemens Herschel Fund	1,126 97	58 45	12 15	–	13 50	1,184 07
Edward W. Howe Fund	1,081 19	56 07	11 66	–	70 82	1,078 10
	$86,354 68	$4,477 57	$931 27	$319 00	$1,716 12	$90,366 40
Current Fund	1,500 00	–	–	8,845 22‡	8,845 22	1,500 00
Total, all funds	$87,854 68	$4,477 57	$931 27	$9,164 22	$10,561 34	$91,866 40

* Net decrease.

† Entrance fees.

‡ Includes transfers of $1,448.85 from the Permanent Fund and $35 from the Alexis H. French Fund to the Current Fund.

TABLE 2. — RECORD OF INVESTMENTS

	Date of Maturity or Classification	Fixed or Current Interest Rate	During the Year March 10, 1936, to March 10, 1937				March 10, 1937		
					Sold or Matured				
			Interest or Dividends Received	Additional Amount Invested	Amount Received	Profit (or Loss)	Par Value or Number of Shares	Book Value	Market Value
Bonds									
Alabama Water Service Co.	Jan. 1, 1957	5 %	$87 50	–	$2,004 20	$89 20	–	–	–
Arkansaw Water Co.	Oct. 1, 1956	5 %	25 70	–	1,000 00	−5 00	–	–	–
Baltimore & Ohio R.R.	Aug. 1, 1939	4½%	29 12	$998 75	–	–	$1,000 00	$998 75	$1,010 00
The Boston Terminal Co.	July 1, 1950	4 %	40 00	–	–	–	1,000 00	1,000 00	1,090 00
California Water Service Co.	Apr. 1, 1958	5 %	43 61	–	1,050 00	77 22	–	–	–
Canadian Pacific R.R.	July 2, 1949	4 %	74 46	5,342 50	–	–	5,000 00	5,342 50	5,400 00
Central Maine Power Co.	Oct. 1, 1960	4 %	80 00	–	–	–	2,000 00	2,035 36	2,050 00
Central Power & Light Co.	Aug. 1, 1956	5 %	50 00	–	–	–	1,000 00	1,005 00	930 00
Chicago & Northwestern Ry. Co.	Nov. 1, 1987	5 %	–	–	–	–	1,000 00	1,102 50	510 00
The Cleveland Union Term. Co.	Apr. 1, 1973	5 %	50 00	–	–	–	1,000 00	998 75	1,077 50
The Cleveland Union Term. Co.	Oct. 1, 1977	4½%	45 00	–	–	–	1,000 00	980 00	1,046 25
Consolidated Gas Co. of N. Y.	June 1, 1951	4½%	105 00	–	2,115 80	115 44	–	–	–
Con. Water Co. of Utica, N. Y.	May 1, 1958	4½%	135 00	–	–	–	3,000 00	3,037 50	2,850 00
Houston Lighting & Power Co.	June 1, 1981	4½%	52 50	–	1,037 50	57 22	–	–	–
Illinois Power & Light Corp.	Dec. 1, 1956	5 %	50 00	–	–	–	1,000 00	981 25	1,015 00
Louisiana Power & Light Co.	Dec. 1, 1957	5 %	50 00	–	–	–	1,000 00	835 00	1,055 00
Mississippi Power & Light Co.	Dec. 1, 1957	5 %	50 00	–	–	–	1,000 00	972 50	942 50
The New Rochelle Water Co.	Nov. 1, 1951	5½%	25 42	–	905 00	−40 00	–	–	–
Pacific Gas & Electric Co.	June 1, 1960	4½%	22 50	–	1,050 00	50 00	–	–	–
Pennsylvania Power & Light Co.	Apr. 1, 1981	4½%	90 00	–	–	–	2,000 00	1,927 63	2,115 00
The Pennsylvania Railroad Co.	June 1, 1965	4½%	45 00	–	–	–	1,000 00	1,017 74	1,085 00
The Pennsylvania Railroad Co.	Apr. 1, 1970	4½%	45 00	–	–	–	1,000 00	971 58	1,027 50
Penn. State Water Corp.	Mar. 1, 1952	5½%	95 33	–	2,040 00	–	–	–	–
Peoria Water Works Co.	Aug. 1, 1950	5 %	50 00	–	–	–	1,000 00	1,010 00	1,000 00
Public Electric Light Co.	Oct. 1, 1956	5½%	55 00	–	2,110 00	135 00	–	–	–

TABLE 2. — RECORD OF INVESTMENTS — *Concluded*

	Date of Maturity or Classification	Fixed or Current Interest Rate	DURING THE YEAR MARCH 10, 1936, TO MARCH 10, 1937				MARCH 10, 1937		
			Interest or Dividends Received	Additional Amount Invested	SOLD OR MATURED		Par Value or Number of Shares	Book Value	Market Value
					Amount Received	Profit (or Loss)			
Bonds—*Con.*									
Southern Pacific Co.	Mar. 1, 1977	4½%	$45 00	–	–	–	$1,000 00	$955 00	$985 00
Southwestern Gas & Electric Co.	Nov. 1, 1960	4 %	40 00	–	–	–	1,000 00	995 00	1,010 00
Tennessee Public Service Co.	Oct. 1, 1970	5 %	50 00	–	–	–	1,000 00	1,002 50	797 50
Texas Electric Service Co.	July 1, 1960	5 %	100 00	–	–	–	2,000 00	2,000 00	2,105 00
The Toledo Edison Co.	Nov. 1, 1962	5 %	100 00	–	–	–	2,000 00	2,085 00	2,155 00
Western Maryland R.R. Co.	Oct. 1, 1952	4 %	40 00	–	–	–	1,000 00	982 78	1,031 25
Total, bonds	– –	–	$1,771 14	$6,341 25	$13,312 50	$479 08	–	$32,236 34	$32,287 50
Co-operative Banks									
Codman Co-operative Bank	Ser. 37, 1941	4 %	$60 60	$2,218 00	–	–	20	$2,218 00	$2,218 00
Merchants Co-operative Bank	Ser. 140, 1940	3½%	174 00	654 00	–	–	40	5,342 80	5,342 80
Suffolk Co-operative Bank	Ser. 134, 1940	4 %	196 89	4,472 00	–	–	40	5,732 00	5,732 00
Suffolk Co-operative Bank	Mat. Shrs.	3 %	30 00	–	–	–	5	1,000 00	1,000 00
Volunteer Co-operative Bank	Ser. 135, 1936	4 %	32 75	47 00	$5,000 00	–	–	–	–
Total, co-operative banks	– –	–	$494 24	$7,391 00	$5,000 00	–	–	$14,292 80	$14,292 80
Stocks									
Alabama Power Co.	Cum. Pref. 7%	$7 00	$70 00	–	–	–	10	$980 00	$840 00
American Tel. & Tel. Co.	Common	9 00	414 00	–	–	–	46	5,346 04	8,090 25
Bankers Trust Co.	Common	2 00	20 00	–	–	–	10	640 00	795 00
Carolina Power & Light Co.	Cum. Pref. 7%	7 00	113 70	–	–	–	10	1,020 00	990 00
Commonwealth & Southern Corp.	Cum. Pref.	3 00	24 00	–	–	–	8	1,019 89	585 00
Commonwealth & Southern Corp.	Common	–	–	–	–	–	25		
Commonwealth & Southern Corp.	Opt. Warrants	–	–	–	–	–	12		

Consolidated Gas Co. of N. Y.	Common	1 00	35 00	–	–	–	20	1,906 50	835 00
Consumers Power Co.	Cum. Pref. 6%	6 00	60 00	–	1,059 20	39 20	–	–	–
Electric Power & Light Corp.	Common	–	–	–	–	–	10	63 75	251 25
General Electric Co.	Common	1 00	52 50	1,571 94	–	–	50	2,341 47	3,062 50
Georgia Power Co.	Cum. Pref.	6 00	63 00	–	1,217 02	73 69	–	–	–
Home Insurance Co.	Common	1 00	12 00	1,245 00	–	–	30	1,245 00	1,185 00
Minnesota Power & Light Co.	Pref. 7%	7 00	87 50	–	–	–	10	980 00	970 00
National Dairy Products	Common	1 20	–	1,154 74	–	–	50	1,154 74	1,175 00
New England Power Assoc.	Pref.	4 00	100 00	–	–	–	20	1,815 00	1,750 00
New England Power Co.	Pref.	6 00	60 00	–	1,299 30	339 30	–	–	–
New York Central R.R. Co.	Common	–	–	–	–	–	10	870 45	536 25
North American Trust Shares	Cum. July 15, '55	248	259 00	1,945 00	–	–	1,500	5,342 00	5,790 00
Northwestern Electric Co.	Cum. 1st Pref.	7 00	70 00	–	–	–	10	980 00	840 00
Pacific Gas & Electric Co.	Cum. 1st Pref. 6%	1 50	90 00	–	–	–	60	1,922 02	2,425 00
Pacific Gas & Electric Co.	1st Pref. 5½%	1 375	27 52	–	–	–	20		
Pacific Gas & Electric Co.	Common	1 50	104 00	–	–	–	64	1,808 79	2,272 00
Penn. Gas & Elec. Corp.	Pref.	7 00	210 00	–	–	–	30	2,955 00	2,820 00
Southern Cal. Edison Co. Ltd.	Cum. Orig. Pref.	1 50	60 00	–	–	–	40	1,161 22	1,560 00
Southern Cal. Edison Co. Ltd.	Common	1 50	32 50	–	–	–	20	539 75	570 00
Tampa Electric Co.	Common	2 24	50 40	1,151 25	–	–	30	1,151 25	1,110 00
The Tenn. Elec. Power Co.	1st Pref.	7 00	70 00	–	–	–	10	950 00	695 00
Timken Roller Bearing	Common	1 50	33 75	1,018 97	–	–	15	1,018 97	1,061 25
Trimount Dredging Corp.	Pref. C.	–	–	–	–	–	2	–	–
United States Steel Co.	Common	–	–	–	–	–	10	860 75	1,252 50
United States Trust Co. of Boston	Cum. Conv. Pref.	80	–	4,837 50	–	–	225	4,837 50	5,062 50
Utah Power & Light Co.	Cum. Pref.	2 91⅔	23 32	–	–	–	10	990 00	725 00
West Penn. Power Co.	Cum. Pref.	7 00	70 00	–	–	–	10	1,030 00	1,210 00
Total, stocks	– –	–	$2,212 19	$12,924 40	$3,575 52	$452 19	–	$44,930 09	$48,458 50

Report of the Secretary

Boston, March 9, 1937.

To the Boston Society of Civil Engineers:

The following is a statement of cash received by the Secretary, and of the expenditures approved by the President, in accordance with the budget adopted by the Board of Government for the year ending March 17, 1937:

	Current Fund Account		Cash Receipts	Credit Transfers by Treasurer	Net Distribution under Budget	
	Account Number	Expenditures			Expenditures	Receipts
Office:						
Secretary — salary and expense	(1)	$240 00	–	–	–	–
Stationery, printing and postage	(2)	340 65	–	–	–	–
Incidentals and petty cash	(3)	123 74	–	–	–	–
Insurance and Treasurer's bond	(4)	40 66	–	–	–	–
Safety deposit box	(5)	11 00	–	–	–	–
Quarters, rent, light, telephone	(7)	1,753 85	$600 00*	–	–	–
Office, clerical	(8)	1,440 00	–	–	–	–
Auditors for 1935 accounts	(9)	50 00	–	–	–	–
Total		$3,999 90	$600 00	–	$3,399 90	–
Meetings:						
Rent of halls, etc.	(11)	$167 65	–	–	–	–
Stationery, printing and postage	(12)	55 53	–	–	–	–
Social activities	(13)	323 40	$150 50†	–	–	–
Stereopticon and reporting	(14) (15)	74 50	–	–	–	–
Annual meeting (March, 1936)	(16)	75 00	–	–	–	–
Total		$696 08	$150 50	–	$545 58	–

Sections:						
Sanitary Section	(21)	$10 00	–	–	–	–
Designers Section	(22)	25 50	–	–	–	–
Highway Section	(23)	11 50	–	–	–	–
Northeastern University Section	(24)	12 25	–	–	–	–
Total		$59 25	–	–	$59 25	–
Journal:						
Editor — salary and expense	(31)	$310 00	–	–	–	–
Printing and postage	(32)	2,460 63	–	–	–	–
Advertising, commission and receipts	(34)	381 75	$890 96	–	–	–
Sale of JOURNALS	(35)	–	276 75	–	–	–
Total		$3,152 38	$1,167 71	–	$1,984 67	–
Library:						
Librarian — salary and expense	(41)	$50 48	–	–	–	–
Periodicals	(43)	59 00	–	–	–	–
Binding	(44)	58 40	–	$35 00‡	–	–
Total		$167 88	–	$35 00	$132 88	–
Miscellaneous:						
Library fines		–	$2 50	–	–	–
Badges for members	(51)	$68 50	6 41	–	–	–
Binding JOURNALS for members	(52)	21 25	55 75	–	–	–
Bank charges	(53)	54	20 25	–	–	–
Total		$90 29	$84 91	–	$5 38	–

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* From Engineering Societies of New England, rent, $600.

† Sale of tickets, annual meeting, 1936.

‡ From French Fund (65), $35.

	CURRENT FUND ACCOUNT		Cash Receipts	Credit Transfers by Treasurer	NET DISTRIBUTION UNDER BUDGET	
	Account Number	Expenditures			Expenditures	Receipts
Engineering Societies of New England	(59)	$679 44	–	–	$679 44	–
Dues from members, B. S. C. E.	(70)	–	$5,358 25	–	–	$5,358 25
Income from Permanent Fund		–	–	$1,448 85	–	1,448 85
Total Current Fund		$8,845 22	$7,361 37	$1,483 85	$6,807 10	$6,807 10
Entrance fees, to Permanent Fund (11 new members, 4 juniors, 1 associate, 23 students, 3 juniors transferred to members, and 11 students transferred to junior)		–	$319 00	–	–	$319 00

The above cash receipts have been paid to the Treasurer, whose receipt the Secretary holds.

Respectfully submitted,

EVERETT N. HUTCHINS, *Secretary.*

Report of the Auditing Committee

Boston, March 17, 1937.

We have examined the records and accounts of the Secretary and the Treasurer of the Boston Society of Civil Engineers and have satisfied ourselves that these have been properly kept and that funds received and expended have been duly recorded. We have examined the securities enumerated by the Treasurer and verified the income from investments. In this examination your committee has been assisted by William J. Hyde, Certified Public Accountant.

The signed reports of the Secretary and Treasurer, to which this statement is attached, are properly reconciled, summarize the transactions and indicate the actual financial condition of the Society.

J. Stuart Crandall,
A. B. Edwards,
Auditing Committee of the Directors of the
Boston Society of Civil Engineers.

Statement of the Certified Public Accountant

March 16, 1937.

Mr. J. Stuart Crandall, *Chairman, Auditing Committee, Boston Society of Civil Engineers, Room 715, Tremont Temple, Boston, Massachusetts:*

Dear Sir: — In accordance with your instructions, I have completed an audit of the financial records of the Society for the fiscal year ended March 10, 1937.

Securities held by the Society as at March 10, 1937, were examined and correctly reflected the changes occurring within the year under review. All coupons for interest due were collected except three of the Chicago & Northwestern Railway Company, total $75, which are in default. Dividends earned were verified and all were correctly accounted for. The records of purchases and sales of securities were in agreement with brokers' statements.

Receipts reported as coming from your Secretary were compared with his records and found to be correctly entered in the Treasurer's accounts.

Co-operative bank earnings were verified and found correct.

All vouchers paid were approved by your President and payment substantiated by examination of cancelled checks.

A verified copy of your Treasurer's report is attached hereto, and summarizes the detailed records of his ledger, which I have carefully checked and found in excellent condition.

Respectfully submitted,

Wm. J. Hyde,
Certified Public Accountant.

Report of the Editor

BOSTON, MASS., January 15, 1937.

To the Board of Government, Boston Society of Civil Engineers:

The JOURNAL for the calendar year 1936 (Volume XXIII) was issued quarterly, in the months of January, April, July and October, as authorized by the Board of Government on December 20, 1933.

During the year 1936 there have been published twelve papers, presented at meetings of the Society and Sections. The Contents and Index for the year 1936 are included in the October issue.

The four issues of the JOURNAL contained 321 pages of papers and discussions, 7 pages of Index, and 28 pages of advertising, a total of 356 pages. An average of 1,238 copies per issue was printed. The net cost was $1,711.16 as compared with $2,091.41 for the preceding year.

Table I includes a comparison of the costs for the JOURNAL from 1932 to 1936, inclusive. In Table II details of cost for Volume XXIII of the JOURNAL for the calendar year 1936 are shown.

Respectfully submitted,

EVERETT N. HUTCHINS, *Secretary.*

TABLE I. — COMPARISON OF COSTS OF JOURNAL, 1932–36, INCLUSIVE

YEAR	Volume	Number of Pages*	Gross Total†	Cost per Page	Net Total‡	Cost per Page	Number of Cuts and Inserts
1932 . . .	XIX	635	$3,906 87	$6 17	$2,600 62	$4 10	108
1933 . . .	XX	331	2,265 51	6 84	1,389 36	4 17	82
1934 . . .	XXI	383	2,966 67	7 74	2,081 37	5 43	126
1935 . . .	XXII	323	2,628 16	8 13	2,091 41	6 47	119
1936 . . .	XXIII	356	2,666 42	7 49	1,711 16	4 80	90

* Includes Advertising Section and Annual Index.

† Gross cost includes editing, printing and mailing.

‡ Net cost equals gross less amounts received for advertising, subscriptions and sale of JOURNALS.

TABLE II. — 1936 JOURNAL, VOLUME XXIII

	Issues of —				
	January	April	July	October	Total
Number of pages:					
Papers and discussion	56	93	99	80*	328
Advertising and advertisers' index	6	7	7	8	28
Total pages	62	100	106	88	356
Number of cuts	14	21	21	34	90
Cost of Journal					
Composition and printing	$307 48	$610 04	$589 10	$408 32	$1,914 94
Cuts	69 42	100 01	31 43	115 09	315 95
Wrapping, mailing and postage	–	–	–	–	106 32
Editing	–	–	–	–	321 21
Copyrighting	–	–	–	–	8 00
Total gross cost of JOURNAL	–	–	–	–	$2,666 42

Receipts from subscriptions and sales of JOURNALS and reprints	$276 95	
Receipts (net) from advertising	678 31	
Total receipts		955 26
Net cost of JOURNAL		$1,711 16

* Includes 7 pages of Index of 1936 JOURNAL.

Report of the Committee on Social Activities

BOSTON, March 17, 1937.

To the Boston Society of Civil Engineers:

The Committee on Social Activities submits the following report for the year 1936–37.

At the five regular meetings held at the Engineers Club during the past year, an average of 74 members and guests per meeting were served with buffet suppers. This average is greater by 7 than for the previous year, and 17 greater than for two years ago. The smallest number served was 40 and the greatest 111. On an average, 31 additional persons attended these meetings following the supper.

Eighty-nine members and guests attended the annual dinner at the Chamber of Commerce Building.

One hundred and twenty-five members and guests made the excursion to the Ware and Swift Rivers Project in June. This excursion was made jointly with the International Conference on Soil Mechanics. Lunch was served at Enfield.

One hundred and fifty-two were served at the Student Night meeting at Tremont Temple in November. This meeting included the Student Chapters of the American Society of Civil Engineers at Harvard, Massachusetts Institute of Technology, Tufts, Rhode Island State and Brown, and the Northeastern University Section of the Boston Society of Civil Engineers. The attendance at the meeting following the supper was 165.

The February meeting at Harvard was a joint meeting with the Northeastern Section of the American Society of Civil Engineers and the Student Chapters. One hundred attended the supper and 950 persons were present at the meeting.

The price of supper at the Engineers Club was increased from 90 cents to $1.25 in September, therefore the Board of Government voted that beginning with the December meeting the charge to members of the Boston Society of Civil Engineers would be increased from 50 cents to 65 cents. They also voted that the charge to others than members of the Boston Society of Civil Engineers would be $1.

Ten meetings were held during the year, as follows:

March — Annual dinner, Chamber of Commerce.
April
May
September } — Regular meetings, Engineers' Club.
October
December
June — Excursion to Ware and Swift rivers.
November — Student Night, Tremont Temple.
January — Walker Memorial, M. I. T.
February — Joint meeting with A. S. C. E., Harvard.

The average attendance at meals for all these meetings was approximately 90. The average attendance at the meetings was slightly over 200. This represents a considerable increase over the average attendance for the previous year. We sincerely hope that the attendance at both suppers and meetings will continue to increase.

Respectfully submitted,

JOHN H. HARDING, *Chairman.*

Report of the Committee on Membership and Publicity

MARCH 17, 1937.

To the Boston Society of Civil Engineers:

The Committee on Membership and Publicity presents the following report on its activities for the year ending with the annual meeting, March 17, 1937.

Through an appropriation by the Board of Government, the Committee was able to have published a brochure outlining the activities and purposes of the Society, which was distributed to all members, to all of the new Associate Members of the Engineering Societies of New England, and to numerous others suggested by various members, in all, about 1,200 copies being distributed.

The report of the Secretary gives statistics of increases and losses in membership and of changes in grade. During the year there have been received a total of 77 applications, either for new membership or advancement in grade, which is equal to about 10 per cent of the membership at the beginning of the year.

Respectfully submitted,

Committee on Membership and Publicity,

Wm. F. Donovan,
H. G. Dresser,
A. B. Edwards,
G. A. Graves,
LeRoy M. Hersum,
H. D. Hurley,
K. C. Reynolds,
F. N. Weaver,
H. P. Eddy, Jr., *Chairman.*

Report of the Committee on Relation of Sections to the Main Society

To the Boston Society of Civil Engineers:

The Committee on Relations of Sections to the Main Society, made up of the four Section chairmen, has held no formal meetings during the year. Its function consisted largely in arranging several joint meetings between the main Society and the Sections.

The By-Laws of the Sections require that eligibility to office and right to vote in a Section be limited to members in the Section. Members of the Society become members of the Section upon making written application to the Executive Committee of the Section. This procedure was desirable when separate notices of meetings of the various Sections as well as of the main Society were sent out. The membership list of the Section became its mailing list. Since notices are no longer sent out separately for each meeting, but are placed in the Journal of the Engineering Societies of New England, the need of a separate mailing list for the Section is no longer required.

Each year when the clerk of the Section makes his report on the activities of the Section he finds that rarely has a member of the main Society made application for membership in a Section — it is no longer needed to receive notices of Section meetings and attend them. Also, in the selection of officers for the Section it frequently happens that the most active members of the Section who would be considered for holding office have not become officially affiliated with the Section.

In order to eliminate needless administrative detail, it is recommended that Article II of the By-Laws of the Sections be revised, as follows:

SECTION 1. Members of the Boston Society of Civil Engineers in all grades shall be members of the Sections except that membership in the Northeastern University Section is limited to members of the Society in all grades, who are students, graduates or members of the faculty of Northeastern University.

SECTION 2. Only members of the Northeastern University Section who are students at Northeastern University are eligible to hold office in that Section.

Respectfully submitted,

For the Committee,

ALBERT HAERTLEIN, *Chairman.*

Report of the Committee on Welfare

MARCH 17, 1937.

To the Boston Society of Civil Engineers:

During the past year there have been no matters relating to the welfare of members of the Society requiring action by the Committee on Welfare, and in consequence it has not been necessary for the Committee to meet during the year. This encouraging situation with regard to the welfare of the members of the Society may be attributed to increased activities in engineering planning and construction through Federal assistance, and to the efforts and accomplishments of the Emergency Planning and Research Bureau, Inc.

The principal activities of the Emergency Planning and Research Bureau, Inc., during the past year have been carried on through its free employment service whereby hundreds of men have been registered for employment and many have secured work not only on Federal relief projects, but also in private industry. During the past year there have been 195 members of the Boston Society of Civil Engineers registered with the Bureau.

All members of the Society who are desirous of securing a position should register with the employment service of the Emergency Planning and Research Bureau, Inc.; this employment service is desirous of securing the registration of all members of the engineering group who are available for employment.

Respectfully submitted,

RALPH W. HORNE, *Chairman.*

Report of the Library Committee

March 17, 1937.

To the Boston Society of Civil Engineers:

The Library Committee has held four meetings during the past year.

Records show that about 125 books were taken out during the year.

Fines have totaled $6.41.

Sale of magazines has netted $2.50.

Through the courtesy of J. B. Babcock, 3d, the copy of Emerson's "Hydrodynamics" has been replaced.

Through the courtesy of Prof. Arthur Casagrande, of the Harvard School of Engineering, a set of three volumes of the "Proceedings of the International Conference on Soil Mechanics and Foundation Engineering," held June 22 to 26, 1936, has been donated to the Society.

A brochure of "The San Francisco–Oakland Bay Bridge" has been donated by Prof. Albert Haertlein.

The following books have been donated by various individuals:

"Men, Money and Molecules," by William Haynes, gift of the Chemical Foundation of New York.
"Industrial America," by Arthur Pound, from Atlantic Monthly Company.
"Living Together in a Power Age," by Samuel S. Wyer, from Associated Press.
"Deterioration of Structures in Sea Water," Committee Report of the Institute of Civil Engineers.

The following reports have been added to the Library:

Technical Procedure for Geodetic Surveys.
Connecticut River Valley Water Resources Bibliography.
High Water Data — Flood of March, 1936, in Massachusetts.
Sources of Pollution — Blackstone River Valley.
Deficiencies in Basic Hydrologic Data.

The following books have been purchased by the committee:

"Boundaries and Adjacent Properties," by R. H. Skelton.
"Engineering Valuation," by Marston and Aag.
"Handbook of Fire Protection," by Crosby-Fiske Foster.
"Hydraulics," by F. C. Lea.
"Materials of Engineering," by H. F. Moore.
"Materials and Structures," by E. H. Salmon.
"Plain Concrete," by E. E. Bauer.
"Resistance of Piles to Penetration," by R. V. Allin.
"Sewage and Sewage Treatment," by W. A. Hardenbergh.
"Soil Erosion and its Control," by Quincy C. Ayres.
"The Steady Flow Traffic System," by Fritz Malcher.
"Water Purification Control," by E. S. Hopkins.

A list of state reports to be discarded will be posted in the rooms of the Society. Any one desiring any of these may have them by applying at the Library.

Respectfully submitted for the Chairman,

A. J. Harty,

By Edw. S. Averell, *Librarian.*

Report of the Committee on John R. Freeman Scholarship Fund

Boston, Mass., March 17, 1937.

To the Boston Society of Civil Engineers:

The John R. Freeman Fund Committee recommends to the Board of Government that the following-mentioned procedure be followed in connection with the administration of the John R. Freeman Fund for the year 1937:

Mr. Leslie J. Hooper, Worcester Polytechnic Institute, Worcester, Mass., was awarded the Freeman Scholarship in June, 1934. He has been making a study and report on the hydraulic laboratories in this country. His inspection work is finished and his report will be ready for printing at an early date. He has done a tremendous amount of work on this assignment, far more than would be expected from a Freeman scholar, and has received the usual amount of $1,800 which accompanies one year's scholarship.

The Committee recommends —

1. A further payment of $500 be made to Mr. Hooper from the Freeman Fund to cover extraordinary expenses in traveling and preparation of his report.

2. That the report of Mr. Hooper be printed as an extra number of the JOURNAL of the Boston Society of Civil Engineers, the number of copies printed being sufficient for the membership of the Society, and for distribution to all of the laboratories who have assisted in the work, to the National Engineering Societies, and others who may desire them, approximately 1,500 copies, the cost of the printing and mailing to be paid for from the Freeman Fund.

The scope of the report has been determined by the Committee and Mr. Hooper.

3. If the report of the Run-off Committee is published, the Freeman Fund Committee would suggest that a portion of the cost of the printing of the report be paid for from the Freeman Fund, the amount of such contribution to be determined when the cost is known.

4. That a Freeman Scholarship be granted for the year 1937–38, and that notice of the same be published in the JOURNAL. Instructions to be sent to applicants for the award of the scholarship are being prepared by the Committee and will be sent to the Secretary for forwarding to the applicants.

Very truly yours,

CHARLES T. MAIN,
Chairman, Freeman Fund Committee.

Report of the Committee on Run-off

BOSTON, MASS., March 17, 1937.

To the Boston Society of Civil Engineers:

The Chairman of the Committee on Run-off offers at this time a preliminary draft of a final report of the Committee on Run-off. It is a report of about one hundred pages, with tables, diagrams, etc.

This Committee was appointed in 1931 to continue the work of the 1922 Committee on Run-off, so that the present report includes fifteen years' records of rain and snowfall and run-off from New England streams. Also these later and more complete records afford the opportunity of testing out many of the conclusions suggested in the earlier report.

This draft of the report was intended to be in the hands of the Steering Committee by January 1 of this year, but it was delayed because of the many reports and statements of a business nature required of me at the beginning of the new year. Copies of the report reached the Steering Committe only about a week before the annual meeting, affording little opportunity to study and criticize the report.

If the Society votes to print the final report of the Committee, opportunity will be given each member of the Committee to study the material of the report and its conclusions.

ARTHUR T. SAFFORD,
Chairman, Committee on Run-off.

Report of the 1936 Flood Committee

BOSTON, March 17, 1937.

The Flood Committee held its first meeting on April 15 and appointed a "steering" subcommittee. This subcommittee found that there were so many agencies collecting, compiling and analyzing flood data that there was nothing useful that the committee could do without duplication of effort. It was decided, however, that a list of all the agencies engaged in flood work would be useful. The Committee therefore prepared such a list which was published in the April JOURNAL of the Society, and later a more complete list of which about seventy copies were sent to members of the Flood Committee, all the flood agencies and others interested. The complete list contained the names of twenty-eight agencies collecting flood data in New England.

Just as in the case of the 1927 flood, the subcommittee found that nothing further useful could be done until various reports containing the flood figures were available. Copies of the United States Army Engineers reports on Flood Control for the New England rivers have been requested, but have not yet been received. It is expected that these or abstracts of them will soon be available for submission to the Committee for any action on these flood control plans that may seem desirable. Other reports will soon be available, so that the Committee can then proceed with its study of the flood. No definite plans for this study have yet been made.

In this connection it seems pertinent to refer to the value to the science of flood hydrology of the work of the 1927 Flood Committee by a quotation from the recently published United States Geological Survey Water Supply Paper No. 771, "Floods in the United States."

The Boston Society of Civil Engineers proposed a formula based on a study of floods in New England. . . .

The idea of a fairly constant base for hydrographs of storm run-off at a given station, with the peak discharge varying with the magnitude of that portion of the rainfall appearing as direct surface run-off during a prescribed period, expressed or implied in connection with the presentation of this formula, is one of the fundamentals underlying some of the most valuable recent contributions to the science of hydrology.

The 1927 Committee has set a high standard of achievement for the 1936 Committee to live up to.

Respectfully submitted,

ARTHUR T. SAFFORD, *Chairman.*

By H. M. TURNER, *Vice-Chairman.*

Report of the Executive Committee of the Sanitary Section

BOSTON, March 3, 1937.

To the Sanitary Section, Boston Society of Civil Engineers:

During the year four meetings have been held, as follows, at which the following papers have been given:

March 4, 1936. — Annual meeting and election of officers at the Society rooms. Adjourned to join meeting of Engineering Societies of New England on Aviation. *Attendance,* 15.

April 1, 1936. — "Chemical Precipitation of Sewage and Industrial Wastes," by Almon L. Fales of the firm of Metcalf & Eddy, Consulting Engineers. *Attendance,* 43.

October 7, 1936. — "The Description of the New Natick Sewage Disposal Plant," by Mr. Elwood W. Ward. *Attendance,* 23.

December 2, 1936. — "Works Progress Administration River Pollution Studies — on the Blackstone, Hoosic, Housatonic and Nashua Rivers," by Mr. Edward Wright, Sanitary Engineer, Massachusetts Department of Public Health, Mr. John H. Harding, Director, W. P. A. State Planning Project, and Mr. Ivan A. Buchanan, Coordinator, W. P. A. State Planning Project. *Attendance,* 37.

The total attendance at all meetings was 118, averaging 30.

The annual outing was held on June 13, 1936. At this time an inspection was made of the Braintree–Weymouth Metropolitan Sewage Pumping Station, also of the Weymouth Water Treatment Plant, the Braintree Water Purification Plant, and the Randolph–Holbrook Water Treatment Plant; 28 persons attended the excursion.

Executive Committee meetings have been held prior to Section meetings.

Respectfully submitted,

For the Executive Committee,

RALPH M. SOULE, *Clerk.*

Report of the Executive Committee of the Designers Section

BOSTON, March 10, 1937.

To the Designers Section, Boston Society of Civil Engineers:

This Section has held eight regular meetings during the year, as follows:

March 11, 1936. — Annual meeting and election of officers at the Society rooms. *Speaker*, Mr. Herman G. Dresser, on "Some Features of Sewer and Culvert Design." *Attendance*, 27.

April 8, 1936. — *Speaker*, Dr. William F. Clapp, Consulting Biologist, on "Marine Borer Problems." *Attendance*, 60.

May 13, 1936. — *Speaker*, Mr. John R. Nichols, Consulting Engineer, on "Unusual Features of Structural Design Involved in the Design of the Harvard Astronomical Observatories." *Attendance*, 14.

October 14, 1936. — *Speaker*, Mr. Irving B. Crosby, Consulting Geologist, on "Engineering Geology of the Passamaquoddy Project." *Attendance*, 45.

November 4, 1936. — *Speaker*, Mr. Henry L. Kennedy, Manager of the Cement Division, Dewey & Almy Chemical Company, on "Practical Application of Catalysis and Dispersion to Cement and Concrete." *Attendance*, 30.

December 9, 1936. — *Speaker*, Dr. J. P. Den Hartog, Associate Professor of Applied Mechanics, Harvard University, on "Dynamics Applied to Engineering Problems." *Attendance*, 150.

January 13, 1937. — *Speaker*, Mr. E. L. Durkee, Assistant Engineer, Fabricated Steel Construction, Bethlehem Steel Company, on "Structural Steel Bearing Piles — Their Use and Capacity." *Attendance*, 85.

February 10, 1937. — *Speaker*, Prof. Dean Peabody, Jr., on "The Effect of the American Concrete Institute's Reinforced Concrete Column Tests on Design." *Attendance*, 40.

Three papers which had been presented at our Section meetings were published in the JOURNAL during the year, being:

"Characteristics of Cohesionless Soils Affecting the Stability of Slopes and Earth Fills," presented by Prof. Arthur Casagrande, November 13, 1935, and published in the January, 1936, number.

"The Design of Statically Indeterminate Trusses," presented by Prof. Albert Haertlein, January 8, 1936, and published in the April, 1936, number.

"The Design Features of Pipe and Concrete Sewers and Culverts," presented by Mr. Herman G. Dresser, March 11, 1936, and published in the October, 1936, number.

The total attendance at all meetings was 451, with an average attendance of 56. For the meetings held in the Society rooms the average attendance has been 43.

Our loss in membership has been exceptionally large this past year, owing to the fact that the main Society has dropped many members for non-payment of dues, and the loss of such members who were also enrolled in the Designers Section is reflected in our roll. There have been some resignations, and we have lost one member by death, Mr. J. W. Woodhall.

Membership in the Section as of March, 1936, was 165. Our loss for the year is 1 member by death and 34 either resigned or dropped, a total loss of 35, leaving our recorded membership for March, 1937, at 130.

Respectfully submitted,

For the Executive Committee,

ANTHONY S. COOMBS, *Clerk.*

Report of Executive Committee of the Highway Section

Boston, February 24, 1937.

To the Highway Section, Boston Society of Civil Engineers:

During the past year four meetings were held.

February 26, 1936. — Annual meeting, election of officers. This was a joint meeting with the Designers Section. The speaker of the evening was Mr. Albert E. Kleinert of the Massachusetts Department of Public Works, who presented a paper on "The Design and Construction of the Quincy–Weymouth Fore River Bridge." *Attendance*, 50.

May 27, 1936. — Mr. E. F. Copell, Traffic Engineer of the Massachusetts Department of Public Works, read a paper on "Methods of controlling traffic." *Attendance*, 30.

November 24, 1936. — Mr. Philip H. Kitfield, Assistant Project Engineer of the Massachusetts Department of Public Works, presented a paper on the subject, "Recent Highway Developments between East Boston Tunnel and Newburyport Turnpike." *Attendance*, 30.

January 27, 1937. — This meeting was held in conjunction with the meeting of the entire Society. The speaker, Mr. Charles R. Noble of the Port of New York Authority, presented a paper on "Design of Modern Express Highways from the Standpoint of Safety." *Attendance*, 75.

The total attendance at all meetings during the year was 185; the average attendance was 46.

Respectfully submitted,

Thomas C. Coleman, *Clerk.*

Report of the Executive Committee of the Northeastern University Section

Boston, March 11, 1937.

To the Boston Society of Civil Engineers:

During the past year the Section has held six meetings, as follows:

March 10, 1936. — "The Manufacture of Cement," by H. W. Eldredge, Technical Service Engineer for Universal Atlas Cement Company. *Attendance* 75.

May 19, 1936. — Annual meeting, election of officers. "The Sanitary Conditions of Boston Harbor," by Dr. Gail P. Edwards, Laboratory Co-ordinator, Massachusetts Department of Public Health. *Attendance*, 75.

October 29, 1936. — "Discussion of Work done by the Massachusetts Department of Public Health," by Francis H. Kingsbury, Senior Assistant Engineer, Massachusetts Department of Public Health. *Attendance*, 27.

January 21, 1937. — "Problems in Design of Waterfront Structures," by Henry Brask, of the George P. Carver Engineering Company. *Attendance*, 29.

February 25, 1937. — "Pile Foundations," by J. Stuart Crandall, President of the Crandall Dry Dock Engineers. *Attendance*, 30.

March 11, 1937. — Special business meeting, election of officers. *Attendance*, 28.

Field trip, October 21, 1936, Division A. — Metropolitan Relief Sewer Project at Stoneham, Medford and Winchester.

Average attendance at meetings was 52.
Average attendance at Field Trips was 45.
Membership in March, 1936, was 95.
Membership at present, 43.

Respectfully submitted,

For the Executive Committee,

ROBERT WM. SMITH, *Clerk.*

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FOUNDED 1848

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